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Vol. 17 No. 1 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **September 1945**

Concrete Construction in the National Forests*

By CLIFFORD A. BETTS†

SYNOPSIS

How U. S. Forest Service applies the fundamentals of good concrete without elaborate control measures to countless small, isolated jobs in the 175,000,000 acres of National forests and along 100,000 miles of roads serving these areas, with pictures to show some of the variety of the work done.

Concrete technique on comparatively small jobs scattered over the 175,000,000 acres of the National Forests or along the 100,000 miles of roads that serve these areas, must of necessity be radically different from that used on a large concentrated dam project or on urban programs near transportation facilities and central mixing plants. The total yardage placed on a large number of minor projects may, and generally does, exceed that of spectacular big jobs. The total value of the small buildings, bridges, and other structures may transcend that of the more publicized ones, yet the opportunity for technical refinements and special equipment is with the big job. Add to this the fact that periodic inspection and maintenance are more common on the major installations, and it becomes evident that on far-flung work programs such as those of the National Forests, the training of hundreds of scattered workers to build durable, fool-proof structures is as difficult as it is desirable.

Nevertheless, progress is being made even on small jobs and discoveries made on the large projects are being applied with success. High-early-strength cement, internal vibrators, moisture control, absorptive form lining, Bentonite-cement grout—all find applications on Forest Service construction.

Simplification of concrete techniques so that any capable field man can produce a dense, workable concrete mix, makes it possible to get

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Fig. 1—Smallwood bridge on Spring Creek, Missouri. It is typical of the low water bridges used where the floodwaters cover so much of the wide valleys that high water bridges would be extremely long and their high cost could not be justified by the traffic. Gage posts on each end of the bridge indicate the depth of the water over the bridge in the early stages of the flood and serve as a guide to the motorist as to his chance of crossing the river.

away from the frequently inapplicable 1:2:4 mix and to design economical mixes with a minimum of effort.

To this end, simple instructions and charts have been carefully worked out without the scientific refinements that are justified on the big job but with the *same basic principles.* The "Water Developments and Sanitation Handbook" of the Forest Service contains complete instructions for producing impervious concrete; the "Truck Trail Handbook" carries specifications for concrete suitable for culverts, bridges, posts, and paved sections; the "Improvement Handbook" includes stucco and plaster data as well as information on reinforced structural concrete. Forms and reinforcement fasteners are described in some detail. It is in this dissemination of handbooks, specifications, and instructions that an organization such as the Forest Service has certain advantages over the individual. There is also the aid rendered by competent inspection and exchange of ideas within the organization.

By providing field forces with laboratory screens and scales to be passed from one job to another, and by making provision for essential

Fig. 2—Inlet of 48 inch pipe culvert at Station 669-65 Douglas City—Peanut Forest Highway, Trinity National Forest, Calif.

Fig. 3—Culvert Intake and drain on Forest Highway, New Hampshire.

control tests by state highway, commercial, or other laboratories so that mistakes will be discovered and not recur, local workers can turn out creditable, long-life structures in large numbers. Men interested in producing good concrete can become proficient in a comparatively short time and then replace the incompetent. Adequate control that will maintain a uniform standard in spite of changes in materials is the major problem on remote jobs which are often finished before standard strength tests can be reported.

Inasmuch as cement and reinforcing steel are covered by mandatory standard government specifications, considerable attention is given to the selection of aggregates. It has been found that it pays to prospect

Fig. 4—Tripoli Bridge, White Mountain National Forest, New Hampshire.

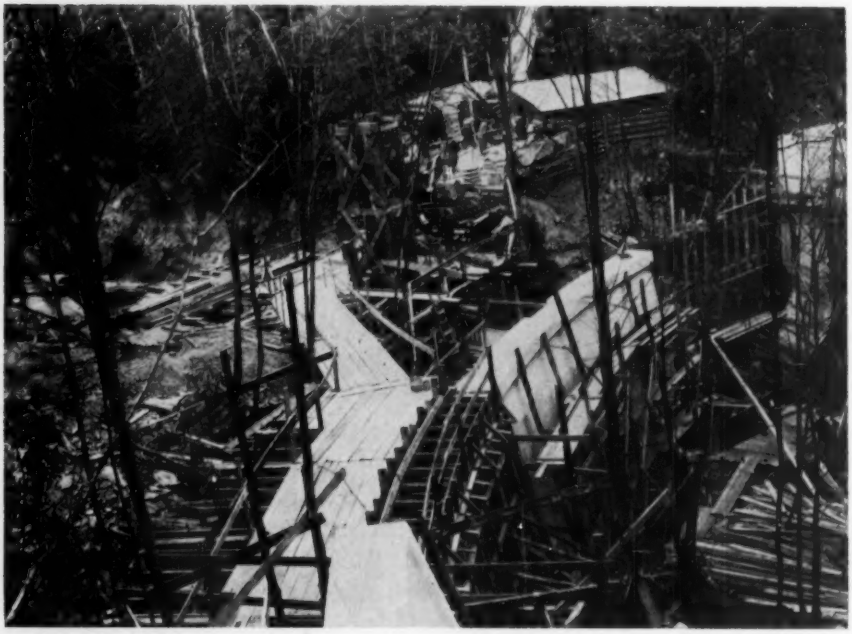

Fig. 5—High Knob Dam, Jefferson Nat'l. Forest, Virginia. Aggregates delivered down hillside by chute to mixer platform; cement stored in log cabin; concrete wheeled to forms; skip serves various levels.

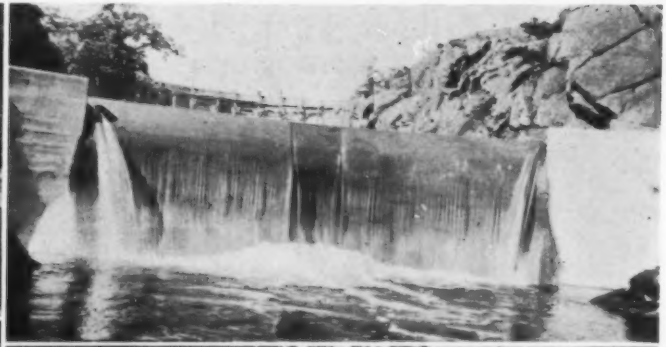

Fig. 6—Green Pastures Dam, Jefferson National Forest, Virginia.

Fig. 7—Herrington Dam side channel spillway, Maryland.

Fig. 8—Quannah Creek dam, Okla.

Fig. 9—Donner Creek Bridge from the West, Tahoe City-Truckee Highway, Tahoe National Forest, Calif.

TABLE 1—BASIC DATA FOR DESIGNING MIX

	Cement	Sand	Gravel	Mixed * Aggregates	Water
Wt. per cu. ft. (dry rodded)	94	104	106	126	62.4
Specific gravity	3.1	2.65	2.65	—	1.0
Fineness Modulus	—	3.05	7.30	5.86	—
% Voids	51.5	36	36	24	—

* 1 part sand to 2 parts gravel.

TABLE 2—COMPUTATIONS FOR MIXES AND YIELDS

	Parts by Vol. (dry rodded)	Absolute Vol. of 1 cu. ft. (lbs. per c.f. ÷ sp. gr. x wt. c.f. water)	Weight lbs.	Absolute Vol. of Mix. (c. f. per 1 bag batch)	Sax. Cem. per c.y. Concrete (27 c. f. divided by abs. vol.)	abs. vol. solids per cu. yd. concrete	Yield = Lbs. per cu. yd. concrete (Wt. x Sax/c. y.)	Parts by wt. (Yield divided by wt. cem.)
Cement	1	$\frac{94}{3.1 \times 62.4}$	= 94	÷ 193 = .49	x 4.58	= 2.24	431	1
Sand	2.5	$\frac{104}{2.6 \times 62.4}$	= 260	÷ 162 = 1.61	x 4.58	= 7.37	1191	2.75
Gravel $\frac{3}{16}$" to 1½"	4.5	$\frac{106}{2.65 \times 62.4}$	= 477	÷ 165 = 2.89	x 4.58	= 13.24	2186	5
Water	* .9	$\frac{62.4}{1 \times 62.4}$	= 56.2	÷ 62.4 = .90	.....	.....	257	....
TOTAL				5.89	27	4.58	22.85	=

$\frac{\text{* 6.75 gals. per sack (1 c.f.)}}{\text{7.5 gals. per cu. ft.}}$ = Water cement ratio

$\frac{\text{Wt. Water}}{\text{Wt. cement}} \times 1.504$ = " " "

$\frac{22.85}{27}$ = .85 = density as placed

TABLE 3—TRIAL CONCRETE MIXES

Guide for Small Jobs Where Tests are not Justified

Estimated Comp. Strength at 28 days (Lbs. per sq. in.)	Max Size Aggregate (Inches)	Sacks Cement Per C.Y. Concrete	Max. Water Per Sack Cement (Gallons)	Lbs. of surface-dry, 2.60 sp. gr. aggregate per sack cement		Mix by Weight
				Fine	Coarse	
2,000+	1	5	8	275	370	1 = 2.9 = 3.9
2,000	2	4½	8	295	430	1 = 3.1 = 4.6
2,000	3	4	8	305	520	1 = 3.2 = 5.5
2,500+	1	5½	7	235	320	1 = 2.5 = 3.4
2,500	2	5	7	245	380	1 = 2.6 = 4.0
2,500	3	4½+	7	255	450	1 = 2.7 = 4.8
3,000+	1	6	6½	215	290	1 = 2.3 = 3.1
3,000	2	5½	6½	225	355	1 = 2.4 = 3.8
3,000	3	5	6½	235	410	1 = 2.5 = 4.4
3,500−	1	6½	6	185	275	1 = 2.0 = 2.9
3,500	2	6	6	195	320	1 = 2.1 = 3.4
3,500	3	5½	6	215+	375+	1 = 2.3 = 4.0
4,000−	1	7+	5½	160−	255	1 = 1.7 = 2.7
4,000	2	6½+	5½	170−	290	1 = 1.8 = 3.1
4,000	3	6+	5½	180−	350	1 = 1.9 = 3.7
4,500−	1	8	5	135	225	1 = 1.4 = 2.4
4,500	2	7½	5	145	265	1 = 1.5 = 2.8
4,500	3	7	5	155	300	1 = 1.6 = 3.2

Use maximum proportion of well-graded coarse aggregate consistent with good workability. To increase slump, reduce the quantities of aggregates, maintaining w/c ratio specified. When specific gravity is not 2.60, multiply pounds of aggregate by ratio of actual specific gravity to 2.60.

all available sources of sand and gravel (pit run or crushed), local, or commercial. In most pits, the overburden, gradation, and structural properties of the materials are fairly easily determined even though some screening of samples or even laboratory tests may be involved. Not only are the standard A.S.T.M. tests for *"Organic Impurities in Sand"* and *"Clay and Silt in Sand"* used, but knowledge acquired through use of local sand, gravel, and rock on roads is also brought into play in recognizing suitable aggregates. It is important to differentiate between dirty, bond-weakening coatings and fine rock powder that fills voids. Frequently the installation of a screening plant or sand washer, or both, is called for and pays dividends. Here is where the portable crushing and screening plants that turn out road materials can serve a dual purpose by also producing aggregates.

Laboratory determinations are usually limited to the minimum; i.e.: 1) Sieve analysis of fine and coarse aggregates; 2) Specific gravity and weight per cubic foot of aggregates; 3) Absorption of aggregates; 4)

Fig. 10—Victoria Dam, Black Hills, South Dakota.

Fig. 11—Two (90-ft.) span rigid frame concrete bridge constructed by Forest Service over E. Branch Pemigewasset River, Swift River Forest Highway, near Lincoln, N. H. White Mtn. Nat'l Forest.

Moisture content of aggregates; 5) Strength tests—(disclosing structural value of aggregates).

Fineness modulus, that big name for a simple, convenient way of designating average size, is used by some field engineers as an indicator of the relative coarseness of the grading. Others use it for aggregate-void mix design, but this is the exception not the rule.

More often envelope curves are developed to fit local materials and grading is kept within these limits. Tables showing the allowable percentages of various sizes of aggregates are also supplied to field men. Permissible tolerance in size variation usually allows considerable latitude in selection. The use of well-graded aggregates is stressed even though it is rarely feasible to separate the coarse aggregate into more than two sizes.

(The author presents Tables 1-3 as reference material for those not experienced in design.)

The choice of maximum size of aggregate follows standard practice except that there is a tendency to utilize "plums" (not closer than 12 in. to each other) where the mixer is unable to handle large aggregate and where the cost of hauling in cement is very high.

Most of the concrete mix design instructions have been built around the water-cement ratio and trial batch methods. In trying to avoid the old pitfall of adding water when the mix is unworkable, it is customary to vary the proportion of sand and cement, retaining the maximum amount of coarse aggregate that is consistent with good workability. This amounts to adding between 3 and 4 pounds of cement for every 1 percent increase in sand.

Water-cement ratios to meet various conditions are tabulated for convenient use. Allowance is, of course, made for moisture in aggregates on most of the jobs, sometimes by weighing, sometimes by empirical rules checked by adding various quantities of water to the dry sand and observing its appearance.

When no facilities are available for proportioning by weight, volumetric measurements are adjusted for bulking. Slumps of 3 to 4 inches predominate.

Cement contents are kept within prescribed limits (say 4 to 7 sacks per cu. yd.) unless written permission to use other amounts is given. Admixtures are rarely used. Data such as that contained in Table 10 "Mixes for Small Jobs," ACI Journal, November 1943, P. 111, are distributed to the field as guides.

Especial care in curing is required as these are non-technical techniques that can be applied in remote localities and have sufficient influence on the ultimate end products to justify the time spent.

Finishing with rough surfaces that harmonize with nature is encouraged; rubbing to a shiny reflecting surface is frowned on.

The *"Handy 40 Rules"* are rough but helpful:

1. Forty per cent of the total dry weight of sand and gravel equals the *weight of sand.*
2. Forty degrees Fahrenheit equals *minimum permissible placing temperature.*
3. Forty seconds equals *minimum mixing time.*
4. Forty minutes equals average *initial setting time.*
5. Forty days equals desirable *curing period.*
6. Forty years equals expected *life of concrete* without maintenance.

Take for example, a typical bridge abutment job in the mountains several miles ahead of finished road grading. Cement may be brought in on pack animals or by trailbuilder and trailer. Sand and gravel may be screened from the stream bed.

A mix is designed by the forest engineer to give the required strength and density utilizing a minimum amount of cement and the best combination of aggregate available. Screen sizes are specified.

Forms of rough lumber (from a nearby mill) supported by hewn logs or poles (cut on the job) give a rough finish, but there is no sloppy concrete, no segregation, no placing in freezing weather without proper precautions.

Concrete resulting from these simple methods of control has such strength and durability that greater refinements would not produce commensurate savings, nor would they be practical on the scattered projects involved. Without the achievements of concrete technicians on major jobs recorded in technical literature as a reference and guide, the success of these relatively rough and ready methods would not be possible. As it is, valuable time and manpower are used where they will do the most good.

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Lapped Bar Splices in Concrete Beams*

By RALPH W. KLUGE†
Member American Concrete Institute
and EDWARD C. TUMA†

SYNOPSIS

An investigation was conducted to determine the general behavior and strength of lapped bar splices which varied in length and method of splicing. The maximum bond resistance developed in the splice and the slip of bar was determined for two types and sizes of reinforcement. The resulting data clearly illustrated the manner in which the stress was transferred from one lapped bar to the other and the relative merits of the two types of bars as well as the effectiveness of the two methods of splicing was shown.

INTRODUCTION

1. Object and scope of investigation

There is little information available in the technical literature concerning the behavior of lapped reinforcing bar splices. As far as is known to the authors, published data pertain to relatively long laps involving plain round bars.‡ It therefore seemed desirable to conduct a study of this type of splice including such variables as length, method of lapping, and type of deformed bar.

Specifically, the purpose of the tests described in this paper was: first, to determine the distribution of stress along the lapped bars and the accompanying bar slip; second, to compare the effectiveness of the lapped bars spaced 1½ bar diameters apart with that of the lapped bars in contact with each other; and, third, to compare the behavior of two types of reinforcing bars, one having a relatively greater lug height than

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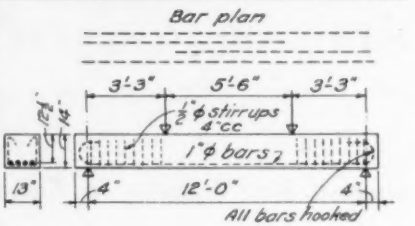
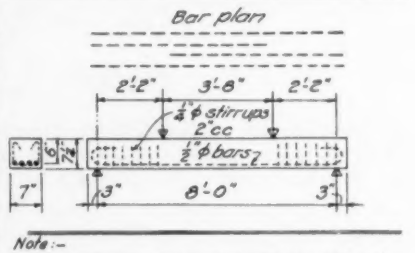
†National Bureau of Standards, Washington, D. C.
‡Technologic Papers of the Bureau of Standards No. 173, "Tests of Bond Resistance between Concrete and Steel," by W. A. Slater, F. E. Richart, and G. G. Scofield, or "Structural Laboratory Investigations in Reinforced Concrete Made by Concrete Ship Section, Emergency Fleet Corporation," by W. A. Slater, Proc. American Concrete Institute, V. XV, 1919.

TABLE 1

	Series	Specimen	Nominal Length of Lap, In.	Design Details of Beams
1" ϕ BARS	A	A-50	50	
		A-40	40	
		A-30	30	
		A-20	20	
		A-40X	40	
		A-30X	30	
		A-20X	20	
	B	B-40	40	
		B-30	30	
		B-20	20	
		B-30X	30	
		B-20X	20	
½" ϕ BARS	A	A-50	25	
		A-40	20	
		A-30	15	
		A-20	10	
		A-40X	20	
		A-30X	15	
		A-20X	10	
	B	B-40	20	
		B-30	15	
		B-20	10	
		B-10	5	
		B-30X	15	
		B-20X	10	
		B-10X	5	

Note:— In specimens designated with the letter "X" following specimen number, lapped bars were in contact with each other. In all other beams, bars were spaced ½ dia. clear.

the other. Both bars were considered to be among the best available as far as bonding properties were concerned.

The test program was divided into two parts as indicated in table 1. One part covered the tests of large beams containing bars 1 inch in diameter, and the other, tests of small beams reinforced with bars ½ in. in diameter. For each size of bar, two sets of tests were made, designated series A and series B. All tests were made on each of the two types of bars.

The tests of series A were primarily concerned with the manner in which the tensile stress or bond stress varied along the lapped bars and the magnitude of the slip at various points. The data also served to compare the behavior of the two types of bars and the relative effectiveness of the two methods of lap splicing. The length of splice in this series varied from 20 to 50-bar diameters and only a single test was made for each length of lap, type and size of bar.

Series B was planned to provide values of average maximum bond stress for several lengths of lap. However, there were few well defined bond failures and, notwithstanding the use of high-yield-strength reinforcement, many specimens failed by yielding of the bars beyond the

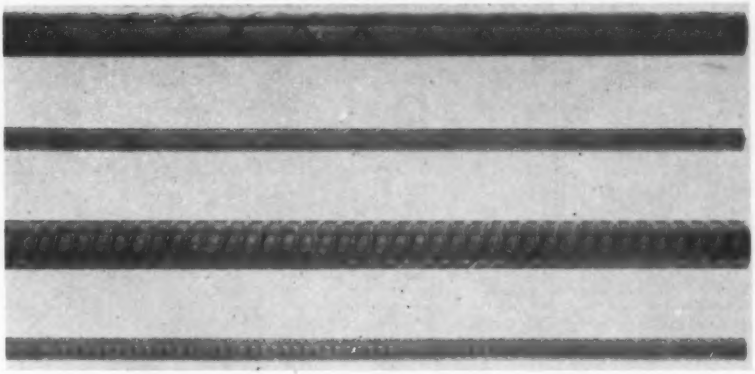


Fig. 1—Reinforcing bars (2 No. 1 at top; 2 No. 2 below)

splice. These tests, however, had some value insofar as indicating the effectiveness of the shorter laps. In this series, the length of the splices varied from 10 to 40-bar diameters and in all but a few instances duplicate tests were made.

DESCRIPTION OF TEST SPECIMENS

2. Design of beams

Details of the test specimens are shown in the diagrams of Table 1. As indicated in the bar plan, each beam contained a single lap splice and two continuous bars. The beams were designed to permit a clear spacing of 1½-bar diameters between all of the bars at a section through the splice and their effective depth was such that theoretically, for a stress of 20,000 psi in the continuous bars, the concrete was stressed to 1,500 psi in compression, assuming a modular ratio of n = 8. This design resulted in a section, outside the region of the lap splice, containing 1.4 percent of tensile reinforcement.

The span of the large beams (containing the 1-in. bars) was 12 ft. and that of the small beam (with the ½ in. bars) was 8 ft. Load was applied to the beams at two points, so spaced as to provide a region of constant moment sufficient in extent to accommodate the longest lap splice of 50-bar diameters. The actual lengths of lap were slightly greater than the given nominal values in order to accommodate an even multiple of a 5-in. spacing of gage holes. The exact values were ¼ in. and ½ in. greater than the nominal lengths for the ½ in. and 1 in. bars, respectively.

The reinforcement consisted of two types of bars, illustrated in Fig. 1, and designated by number. Bar No. 1 had average lug heights of 0.020 in. and 0.027 in., whereas Bar No. 2 had average lug heights of 0.031 in. and 0.063 in. for the ½ in. and 1 in. bars, respectively. The approximate bearing areas of the lugs per lineal inch are given in Table 2. For the

TABLE 2—DIMENSIONS OF REINFORCEMENT

Type	Bar size	*Diameter	*Area	Av. lug height	Lugs per yd.	Approximate bearing area of lugs per lineal inch
	in.	in.	sq. in.	in.		sq. in. per in.
Bar No. 2	½	0.49	0.190	0.031	128	0.16
	1	1.00	0.787	0.063	80	0.42
Bar No. 1	½	0.51	0.198	0.020	100	0.08
	1	1.00	0.785	0.027	50	0.11

*As determined by weight-length method.

TABLE 3—MECHANICAL PROPERTIES OF REINFORCEMENT

(Each value is the average of three tests)

Test Series	Type	Bar size	Yield point	Tensile strength	Elongation in 8 in.
		in.	psi.	psi.	percent
A	Bar No. 2	½	50,000	79,800	20.4
	Bar No. 1	½	61,800	90,000	22.0
	Bar No. 2	1	45,000	71,600	26.2
	Bar No. 1	1	51,600	97,200	18.8
B	Bar No. 2	½	61,800	97,400	16.6
	Bar No. 1	½	55,000	92,600	18.0
	Bar No. 2	1	60,400	94,200	18.8
	Bar No. 1	1	53,000	92,300	23.1

specimens of series A, the reinforcing bars were of intermediate grade or better, whereas the bars used in beams of series B were exclusively of high-strength steel, having a yield point of from 53,000 to 62,000 psi. Their dimensions and mechanical properties are listed in Tables 2 and 3, respectively.

The concrete was designed for a compressive strength of 4,500 psi at 28 days and consisted of moderate-heat-of-hardening portland cement and washed Potomac River sand and gravel, the latter having a maximum size of ¾ in. The proportions of cement, sand, and gravel in the mix were, respectively, 1:2.5:3.3 by dry weight; and the net water content, 7½ gal. of water per sack of cement, corresponding to a water-cement ratio of 0.67 by weight. The resulting concrete had an average slump of 3½ in. and developed an average compressive strength of 4,700 psi tested dry at 28 days.

3. Construction of specimens

Four beams were generally cast at one time, two small beams and two large beams, both pairs containing splices of equal length in terms of the

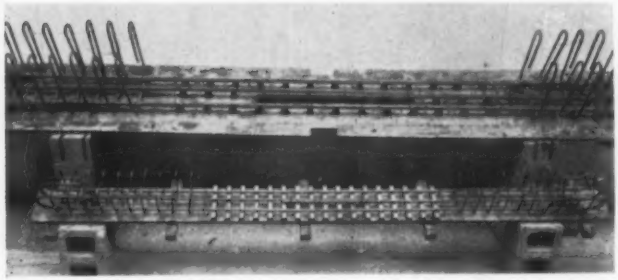


Fig. 2—Reinforcement for 30-bar diameter splice. Lapped bars in contact. Specimen type A-30X.

bar diameter. One of each pair contained Bar No. 1 and the other Bar No. 2. The reinforcement was rigidly supported on wire chairs as well as on tapered wooden inserts. Removal of the latter from the cast beam provided openings to the bars for strain measurements. Spacing of the inserts is shown in Fig. 2 for beams of series A. The beams of series B contained gage holes, similarly spaced, but confined to the region outside the limits of the splice.

The free ends of the lapped bars were saw cut in order to avoid the distorted ends generally accompanying shear cuts. Where no space was provided between the lapped bars, they were firmly wired together at the center and near the ends of the lap with soft iron wire. The concrete was vibrated into place in the beams and the control cylinders. Twenty-four hours after casting, the side forms of the beams and cylinders were removed. The test specimens were then wrapped with several layers of wet burlap and cured in this condition for a period of 7 days, after which they were permitted to dry in the laboratory for 21 days before testing.

DESCRIPTION OF TESTS

4. Procedure

To facilitate the measurement of strain, the beams were tested in an inverted position on standard beam supports resting on the platen of a 600,000-lb. capacity hydraulic testing machine equipped with a 30,000-lb. capacity gage. The load was applied near the ends of the specimens through a pair of wide flange I-beams resting on rollers and bearing plates, which can be observed in Fig. 3 and 4. The weight of the loading beams and auxiliary equipment was approximately 2,400 lb. and was considered as part of the total applied load.

Strain-gage readings were obtained with a 5-in. Whittemore gage for 8 to 10 increments of load. In the tests of series A, strain measurements were made at $2\frac{1}{2}$ in. and 5 in. intervals on the $\frac{1}{2}$ in. and 1 in. bars, respectively, the intervals extending from the free ends of the lapped bar to the load points and at a number of selected points on the continuous

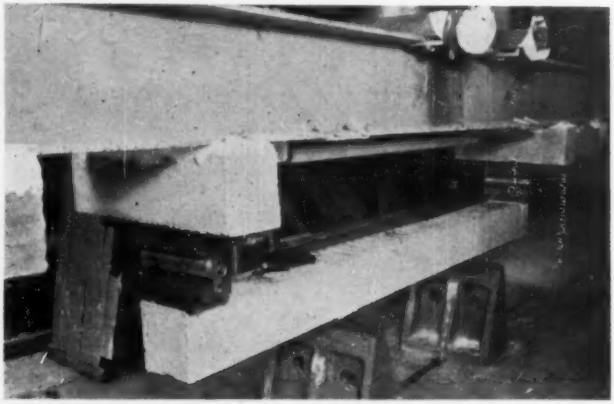


Fig. 3—Eight-foot beam of series A under load.

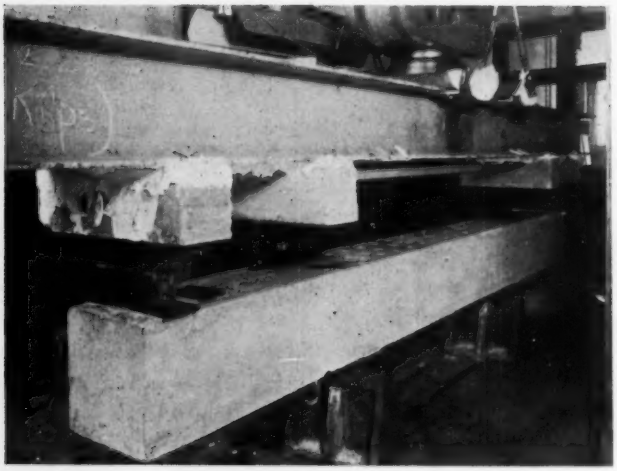
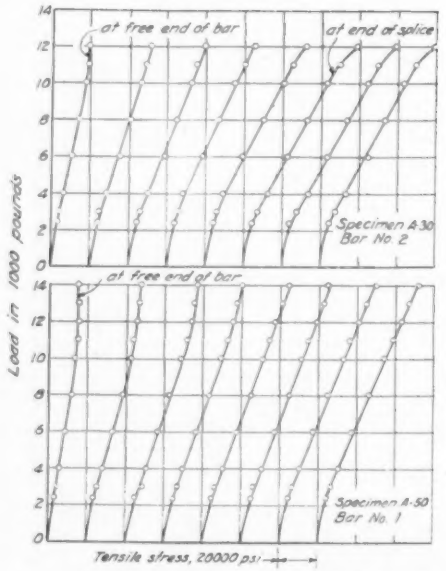
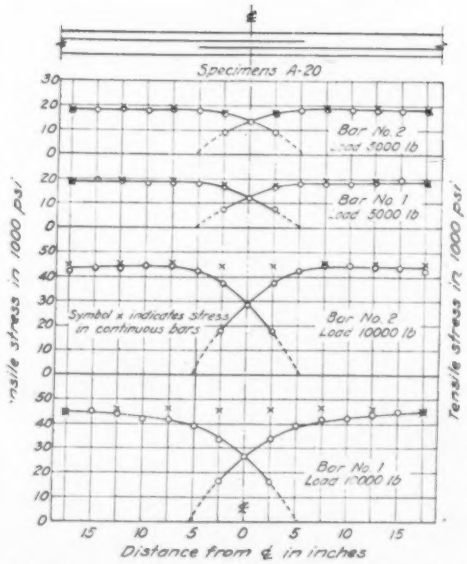


Fig. 4 — Twelve-foot beam of series A under load.

bars. Slip of the lapped bars was determined with the strain gage by observing the relative movement between gage holes on the lapped bars and those on the continuous bars one gage length removed. The readings were corrected for strain in the bars as well as for the angle of the line of measurement. Measurements of strain in the tests of series B were confined to gage lines outside the region of the splice, thus avoiding the uncertain effect of openings in the concrete on the bond stresses along the lapped bars. The average bond stress could be determined from the observed tensile stress in the bars at the ends of the lap.

During the latter stages of the test, the beams were carefully scrutinized for possible longitudinal cracks which, in some beams, formed in the concrete along the lapped bars when the maximum load was reached.





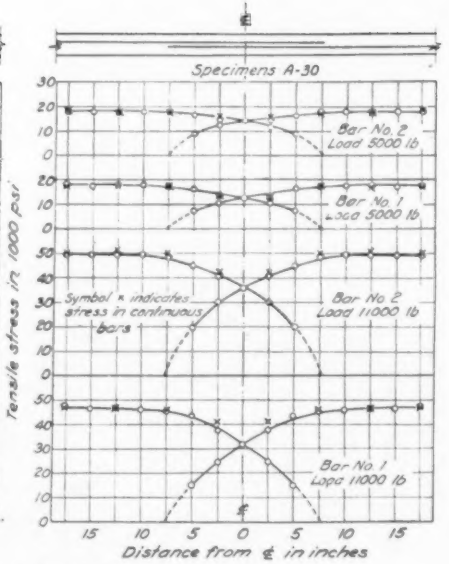


Fig. 5 (left)—Typical load-stress curves for successive gage lines along lapped bars.

Fig. 6 (center)—Distribution of stress along lapped bars—Length of splice 20-bar diameters. ½ in. bars.

Fig. 7 (right)—Distribution of stress along lapped bars—length of splice 30-bar diameters. ½ in. bars.

Stress values are the product of the modulus of elasticity and observed strains.

TEST RESULTS OF SERIES A

5. Distribution of stress along lapped bars

The strains at corresponding gage lines on each of the two bars of the lap were averaged and converted to stress on the assumption that E_s was 30,000,000 psi, and load-stress diagrams plotted for all of the gage points along the bar. Typical diagrams are shown in Fig. 5. The curves are arranged in the order that the gage lines appear on the lapped bar, starting at the free end. After initial cracking of the concrete, the stress in the bar at various points was closely proportional to the load until either the bond stress within the gage line approached a maximum or the yield point of the steel was reached. The curves for the portion of the bar near the free end clearly show the maximum tensile stress, which is also a measure of the maximum bond stress that the bar was capable of developing within the first few inches of embedment.

The variation of tensile stress along the lapped bars of the various specimens is shown in Fig. 6 through 13. The plotted values were taken from the load-stress curves described above for loads near the maximum and for loads which produced a stress of about 18,000 to 20,000 psi in the bars outside the region of the splice. The pair of curves shown on each axis, one for each bar of the splice, are identical except that they are turned end for end. Both the "No. 1" and "No. 2" bars are represented in each figure.

The slope at any point on the curves is obviously a measure of the bond stress developed between the bar and the concrete at that point. The greatest bond stress was developed at the free end of the bar, and this stress decreased fairly uniformly in the 20- and 30-diameter laps over the entire splice. In the longer splices, however, the bond stress, after a similar decrease reached a low value within the middle third, then increased towards the end of the lap. In some instances, where the stresses at the loaded end were less than 30,000 psi, the bars within the middle third of the splice did not develop any bond stress. This was particularly evident in the 40-diameter splices and the 50-diameter splices of the "No. 2" bars. From the standpoint of efficiency, such a splice is longer than necessary.

The stress in the continuous bars at various points in the region of the splices is also indicated on the diagrams by a small cross. Except for splices 20-bar diameters long, the effect of the lapped bars was to decrease greatly the stress in the continuous bars within the limits of the lap, in many instances very closely approaching the stress in the portion of the lapped bar from the center of the lap to its loaded end. In beams containing the 20-diameter splices, the stress in the continuous bars was, in general, fairly constant over the entire lap, the effective steel area of

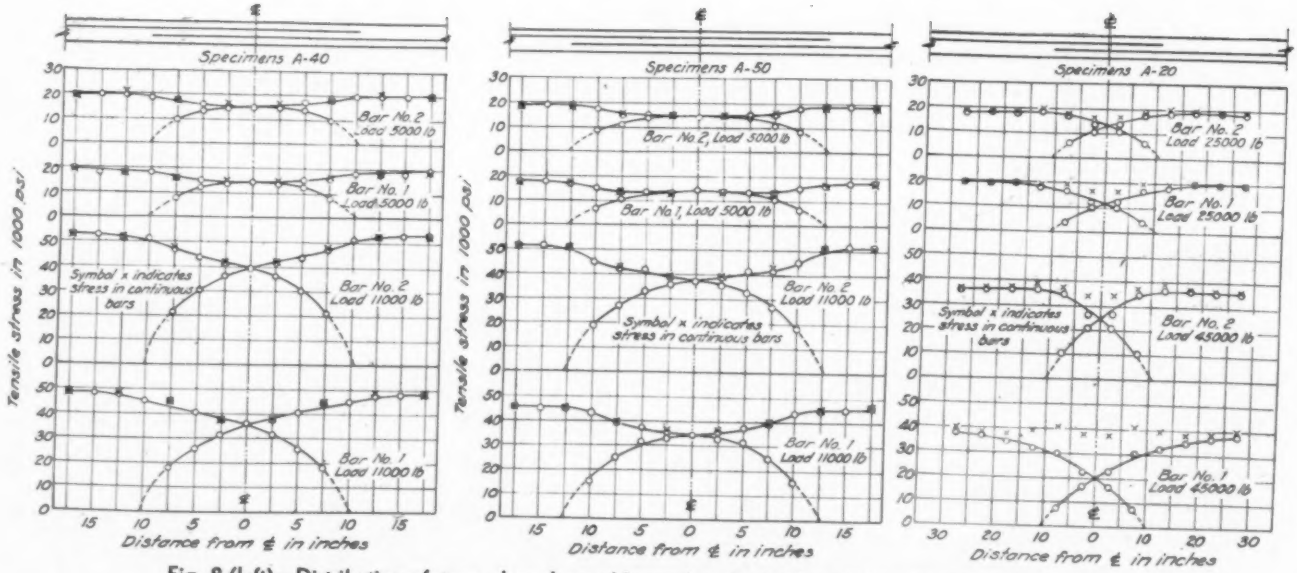

Specimens A-40
Tensile stress in 1000 psi
Bar No. 2 Load 5000 lb
Bar No. 1 Load 5000 lb
Symbol x indicates stress in continuous bars
Bar No. 2 Load 11000 lb
Bar No. 1 Load 11000 lb
Distance from ℄ in inches
Specimens A-50
Bar No. 2, Load 5000 lb
Bar No. 1, Load 5000 lb
Bar No. 2 Load 11000 lb
Symbol x indicates stress in continuous bars
Bar No. 1 Load 11000 lb
Distance from ℄ in inches
Specimens A-20
Bar No. 2 Load 25000 lb
Bar No. 1 Load 25000 lb
Symbol x indicates stress in continuous bars
Bar No. 2 Load 45000 lb
Bar No. 1 Load 45000 lb
Distance from ℄ in inches

Fig. 8 (left)—Distribution of stress along lapped bars—length of splice 40-bar diameters. ½ in. bars.

Fig. 9 (center)—Distribution of stress along lapped bars—length of splice 50-bar diameters. ½ in. bars.

Fig. 10 (right)—Distribution of stress along lapped bars—length of splice 20-bar diameters. 1-in. bars.

Stress values are the product of the modulus of elasticity and observed strains.

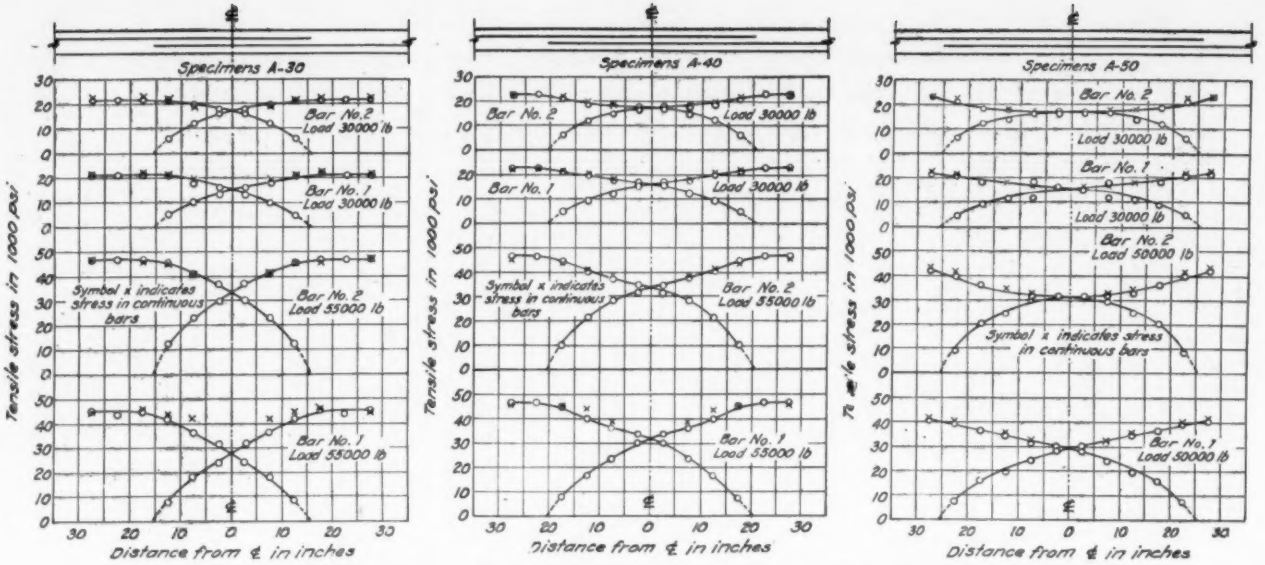
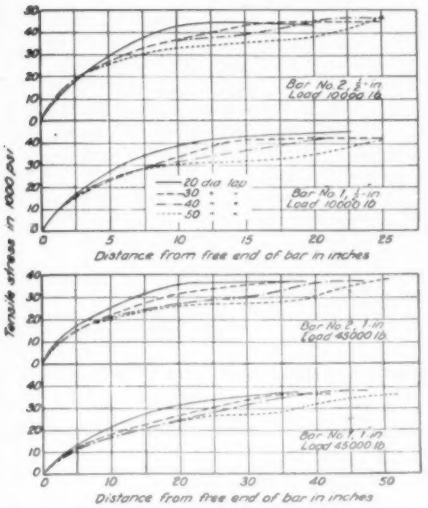

Specimens A-30
Tensile stress in 1000 psi
Bar No. 2 Load 30000 lb
Bar No. 1 Load 30000 lb
Symbol x indicates stress in continuous bars
Bar No. 2 Load 55000 lb
Bar No. 1 Load 55000 lb
Distance from ℄ in inches
Specimens A-40
Tensile stress in 1000 psi
Bar No. 2
Load 30000 lb
Bar No. 1
Load 30000 lb
Symbol x indicates stress in continuous bars
Bar No. 2 Load 55000 lb
Bar No. 1 Load 55000 lb
Distance from ℄ in inches
Specimens A-50
Tensile stress in 1000 psi
Bar No. 2
Load 30000 lb
Bar No. 1
Load 30000 lb
Bar No. 2 Load 50000 lb
Symbol x indicates stress in continuous bars
Bar No. 1 Load 50000 lb
Distance from ℄ in inches

Fig. 11 (left)—Distribution of stress along lapped bars—length of splice 30-bar diameters. 1 in. bars.

Fig. 12 (center)—Distribution of stress along lapped bars—length of splice 40-bar diameters. 1 in. bars.

Fig. 13 (right)—Distribution of stress along lapped bars—length of splice 50-bar diameters. 1 in. bars.

Stress values are the product of the modulus of elasticity and observed strains.



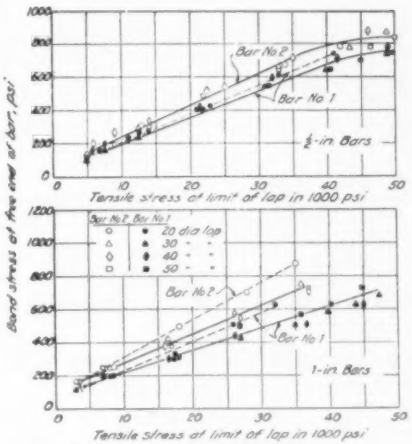

Bar No 2
Bar No 1
½-in. Bars
Bar No 2 Bar No 1
20 dia lap
30 " "
40 " "
50 " "
1-in. Bars
Bond stress at free end of bar, psi
Tensile stress at limit of lap in 1000 psi

Fig. 14 (left)—Distribution of stress along lapped bars for various lengths of splice.

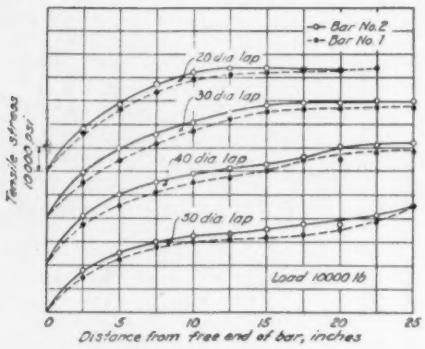
Fig. 15 (right)—Relation between bond stress at free end of bar and tensile stress at limit of lap.

Stress values are the product of the modulus of elasticity and observed strains.

the lapped bars apparently being equivalent to a single bar at any section.

A detailed comparison between the stress distribution curves for each length of lap is shown in Fig. 14. The curves represent the stress distribution for a load of 10,000 lb. on the beams containing the ½ in. bars and 45,000 lb. for the beams containing the 1-in. bars. These loads stressed the reinforcement beyond the splice to 45,000 psi and 37,000 psi, respectively. The slopes of the curves in the vicinity of the origin, which represents the free end of the bar, are almost identical for each set of curves, consequently the bond stress developed within the first few inches of bar embedment was practically alike, regardless of the length of splice for a given load and type of bar.

It is to be expected that some relationship exists between the bond stress at the free end of the lapped bar and the tensile stress in the bar at the other limit of the splice. Fig. 15 indicates, within certain limits, a linear relationship which, for each type of bar, apparently is independent of the length of splice for laps of 30-bar diameters or more. Thus, the data for the 30-, 40- and 50-diameter laps appear to group themselves about individual straight lines, one for Bar No. 1 and one for Bar No. 2, between tensile stresses of 5,000 psi and 40,000 psi. The 20 diameter lap splices also show a similar relationship, but the slopes of the lines are, in some instances, greater than those of the longer splices. Where they differ, the data for the 20 diameter lap are indicated by a broken line.



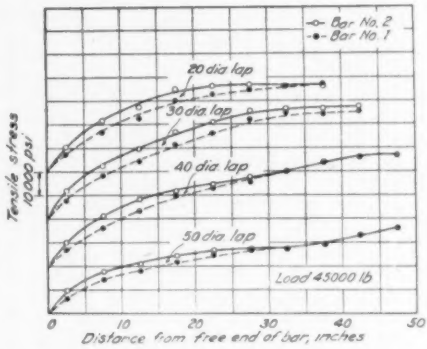


Fig. 16 (left)—Distribution of stress along lapped bars compared for "No. 1" and "No. 2" ½ in. bars.

Fig. 17 (right)—Distribution of stress along lapped bars compared for "No. 1" and "No. 2" 1 in. bars.

Stress values are the product of the modulus of elasticity and observed strains.

The distribution of tensile stresses along the lapped bars for a given load and for various lengths of lap is compared in greater detail for both types of bar in Fig. 16 and 17. In every instance, the No. 2 bars picked up stress more rapidly at their free ends than the No. 1 bar, although the rate of increase of stress beyond an embedment of 5-bar diameters was not significantly different. In a well designed splice, the *normal* stress, that is, the stress the bar would normally carry if it were continuous, should be reached at the end of the splice. In Fig. 18 and 19, the observed stresses in the lapped bars at the end of the splice are plotted with respect to the load applied to the beams for each length of lap. Also shown by solid lines is the load-stress relationship that would obtain if the bars were continuous. This *normal* stress was assumed to be the same as the average stress observed in both the continuous and lapped bars well outside the limits of the splice. The deviation of the observed stress from the *normal* stress beyond a value of about 35,000 psi is evident for most of the beams reinforced with No. 1 bars. The exceptions are the 50-diameter lap and possibly the 40-diameter lap. On the other hand, with the exception of the 20-diameter lap, the observed stresses in the No. 2 bars are practically identical with the *normal* stresses up to the yield point of the steel, which had a value of about 50,000 psi and 45,000 psi for the ½ in. and 1-in. bars, respectively.

It is evident that, if the stress in the lapped bar at the end of the splice is not equal to the *normal* stress, the adjacent continuous bars assume a greater proportion of the total tensile stress. Actually, this was not a serious matter in the beams which were tested with laps greater than 20 diameters; for even at the higher stresses the difference between the actual and *normal* stress was not more than 8 to 10 percent, probably half of

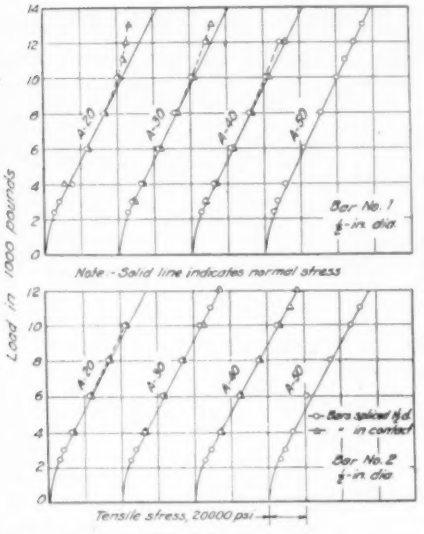


Fig. 18 (left)—Load-stress relation for lapped bars at limits of the splice—Beams of series A. ½ in. bars.

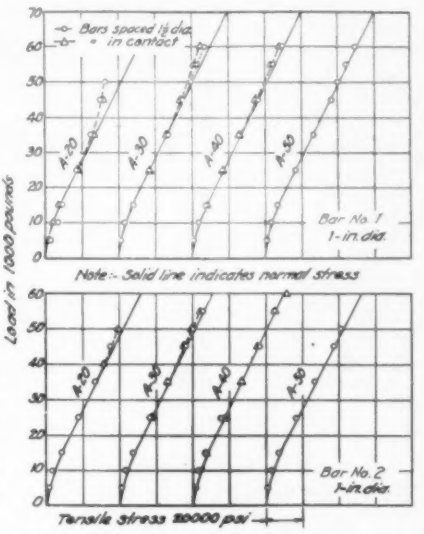


Fig. 19 (right)—Load-stress relation for lapped bars at limits of the splice—beams of series A. 1 in. bars.

Stress values are the product of the modulus of elasticity and observed strains.

which was assumed by the two continuous bars at that point. Consequently, the load which would normally stress the reinforcement, beyond the splice, to its yield point was not materially affected by the slight shifting of stress. This was not true, however, of the beams containing the 20-diameter lap splices, where the load at which yielding of the reinforcement occurred as a rule was somewhat lower than for the other beams.

6. Bond strengths

The load-stress curves for the gage line nearest the free ends of the lapped bars indicated that the tensile stress there reached a maximum value at approximately the same load in all of the specimens tested. Also, the maximum stresses at this point were reasonably alike for a given type and size of bar regardless of the length of lap splice. However, there was a marked difference in the values for the two types of bars. Using the maximum tensile stresses scaled from the curves, the bond strength developed by the lapped bars for an embedded length of about 3 in. was computed for each specimen and the bond stresses thus obtained are listed in Table 4. The values are somewhat higher than would be expected from standard pull-out tests, but compare favorably with results from unpublished data of similar bars in beam tests.

TABLE 4—MAXIMUM BOND STRESS DEVELOPED AT FREE END OF LAPPED BARS

(Av. stress for an embedment of approx. 3 in.)

Specimen	½-in. bars		1-in. bars	
	No. 2	No. 1	No. 2	No. 1
A-50	850	745	880	755
A-40	970	840	960	655
A-30	880	810	1,010	705
A-20	890	745	1,010	755
Average	895	785	965	718
*A-40x	880	675	755	630
*A-30x	920	675	1,010	755
*A-20x	950	810	1,000	760
Average	917	720	922	715

*Lapped bars in contact with each other.

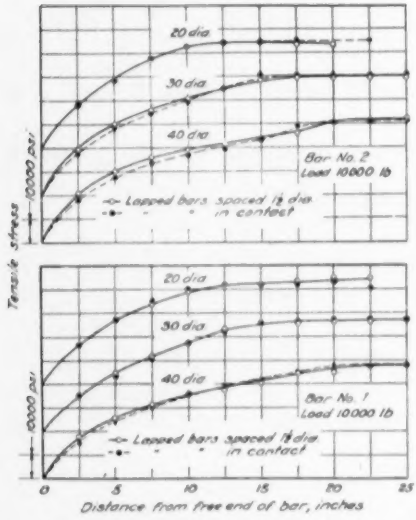


Fig. 20 (left)—Distribution of stress along the lapped bars of two types of lap-splices. ½ in. bars.

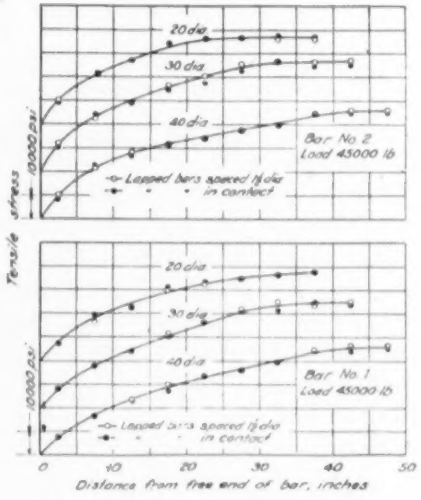


Fig. 21 (right)—Distribution of stress along lapped bars of two types of lap-splices. 1 in. bars.

Stress values are the product of the modulus of elasticity and observed strains.

7. Comparison between methods of lapping

The results of the two methods of lapping, one in which the lapped bars are spaced 1½ diameters clear and the other where they are in contact, are compared in Fig. 20 and 21. Here the stress distribution curves are shown for a given load and for each length of splice. There is practically no difference in the stress between the curves representing each type of lap at any point for the 1-in. bars and little difference for the ½-in. bars.

These differences are probably no greater than would be obtained in duplicate tests. A further comparison is shown in Fig. 18 and 19, in which the load-stress curves for the lapped bars at the limits of the lap are shown for both types of splice. Here, again, the differences are small and in most instances negligible. It is also evident from Table 4 that it made little difference in the bond strength in these tests whether the lapped bars were spaced 1½ diameters apart or in contact.

8. Distribution of slip along lapped bars

There were large differences in slip between certain specimens and these differences were not always consistent. It is highly probable that wide variations in slip data, obtained in the manner described herein, would also be observed in duplicate tests. It should be noted that the slip at each point along the lapped bar was reckoned from a point directly opposite it on the adjacent continuous bar, and consequently does not necessarily indicate the relative movement of the bar with respect to the concrete.

Slip starts, quite naturally, at the free end of the bar where the bond stress is always the greatest and progresses along the bar as the tensile stresses increase at the loaded end. In this respect, the behavior of the bar differs from that in the usual pull-out or beam tests for bond. The distribution of the slip along the lapped bars for various lengths of splice is shown in Fig. 22 and 23. The data are plotted for two loads, one of which produced a tensile stress of approximately 18,000 psi and the other about 40,000 psi at the loaded ends of the lapped bars. Most of the evidence seems to indicate that the distribution and magnitude of the slip is independent of the length of the splice. There appears to be a serious departure from this principle with the No. 1 bars at higher loads. The discrepancy is not readily accounted for except in so far as a part of the variation might be accidental. Within the range of working stresses, however, the length of lap seems to have little or no effect upon the magnitude of the slip for that bar.

With one exception, there was little difference in slip between the No. 1 and No. 2 ½-in. bars for a given tensile stress at the loaded ends of the lapped bars. The exception was the 20-diameter lapped No. 1 bar which showed a considerably greater slip than any of the other lapped bars of this size. The 1-in. No. 1 bars, however, exhibited a decidedly greater slip for a given load or end stress than the 1-in. No. 2 bars or either of the ½ in. bars. The only explanation that can be offered for the dissimilar behavior of the two sizes of No. 1 bars is the possible effect of the lug height. Table 3 shows that the 1-in. No. 1 bar had a much lower ratio of lug height to diameter of bar than the ½-in. bar of the same make, whereas this ratio for the two sizes of No. 2 bars was almost alike.

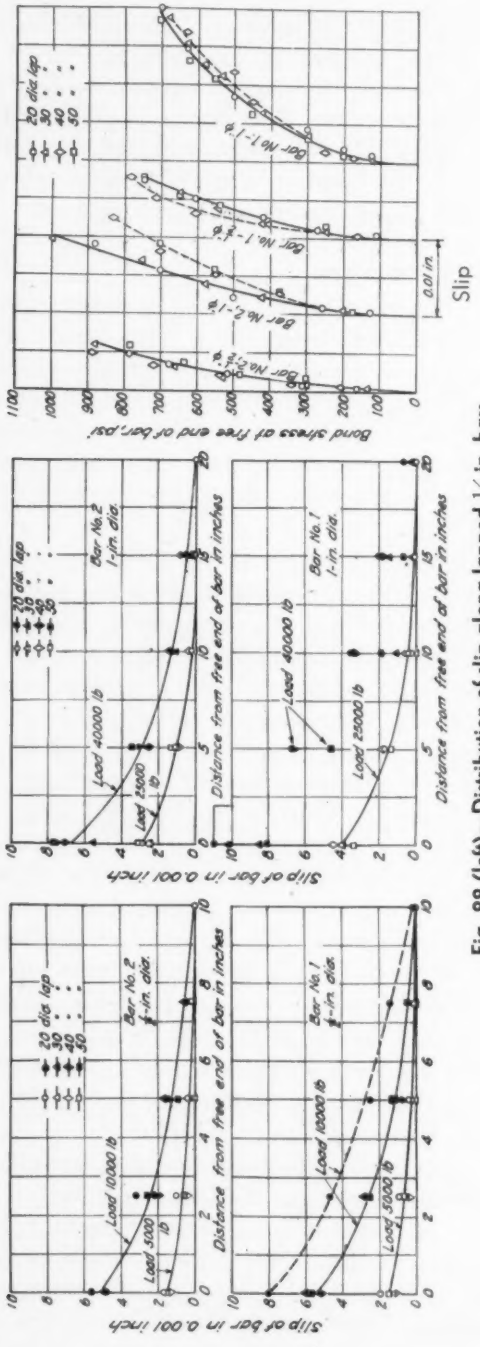

Bond stress at free end of bar, psi
Bar No. 1 - 1 in.
Bar No. 1 - ¾ in.
Bar No. 2 - 1 in.
Bar No. 2 - ¾ in.
0.01 in.
Slip
20 dia lap
30
40
50
Slip of bar in 0.001 inch
Load 40000 lb
Load 25000 lb
Bar No. 2 1-in. dia.
Bar No. 1 1-in. dia.
Distance from free end of bar in inches
Load 10000 lb
Load 3000 lb
Bar No. 2 ¾-in. dia.
Bar No. 1 ¾-in. dia.

Fig. 22 (left)—Distribution of slip along lapped ½ in. bars.

Fig. 23 (center)—Distribution of slip along lapped 1 in. bars.

Fig. 24 (right)—Relation between slip at free end of bar and bond stress near free end.

The difference in the slip behavior of the two types and two sizes of bars is more clearly illustrated in Fig. 24 which shows the relation between slip at the free end of the lapped bar and the bond stress developed within the first few inches of the free end. In the range of bond stresses above 400 psi, the end slip of the ½-in. No. 2 bar is somewhat less than the 1-in. bar of the same make and as previously noted the slip of the ½-in. No. 1 bar is very much less than the 1-in. No. 1 bar for all bond stresses.

It is to be noted that slip is not directly proportional to the bond stress developed by the bar, but apparently is some exponential function of this stress, the expression for the relationship depending upon the type of bar.

TEST RESULTS OF SERIES B

9. General manner of failure

As stated in Section 3, the tests of series B were planned to determine the average maximum bond stress developed by the lapped bars for various lengths of lap. Although the specimens contained high-yield-strength reinforcement, many of the beams failed in tension. All of the 30 and 40-diameter lap splices developed the yield strength of the bar beyond the ends of the lap and all of the 10-diameter splices failed in bond. On the other hand, about half of the 20-bar diameter splices failed in tension and the remainder failed in bond. Many of the bond failures were not well defined because, in some instances, there was a gradual shifting of stress from the lapped bar to the continuous bars as the bond strength was reached. Some of the bond failures, however, were very evident from an examination of the load-stress curves of the lapped bars outside the limits of the splice. The strain in the bar at that point reached a maximum, as shown in Fig. 25 and 26 and either remained at that value or decreased with an increase in load. The breakdown of bond was also reflected, to some extent, by a change in the slope of the load-strain curves for the continuous bars at this section of the beam.

10. Average bond strength

Table 5 lists the maximum average bond stress developed by the various lapped bars. The values are based on the maximum tensile stress reached at a point just beyond the limits of the splice and on the total length of bar embedment, which was considered to be the distance from the free end of the bar to the mid-point of the first gage line.

The characteristic increase in the average bond strength for decreasing lengths of bar embedment is indicated in the table. Obviously, the bond stresses given for those specimens which failed in tension are not necessarily the average maximum values the bars are capable of developing for their particular length of embedment. Where bond failures are indicated,

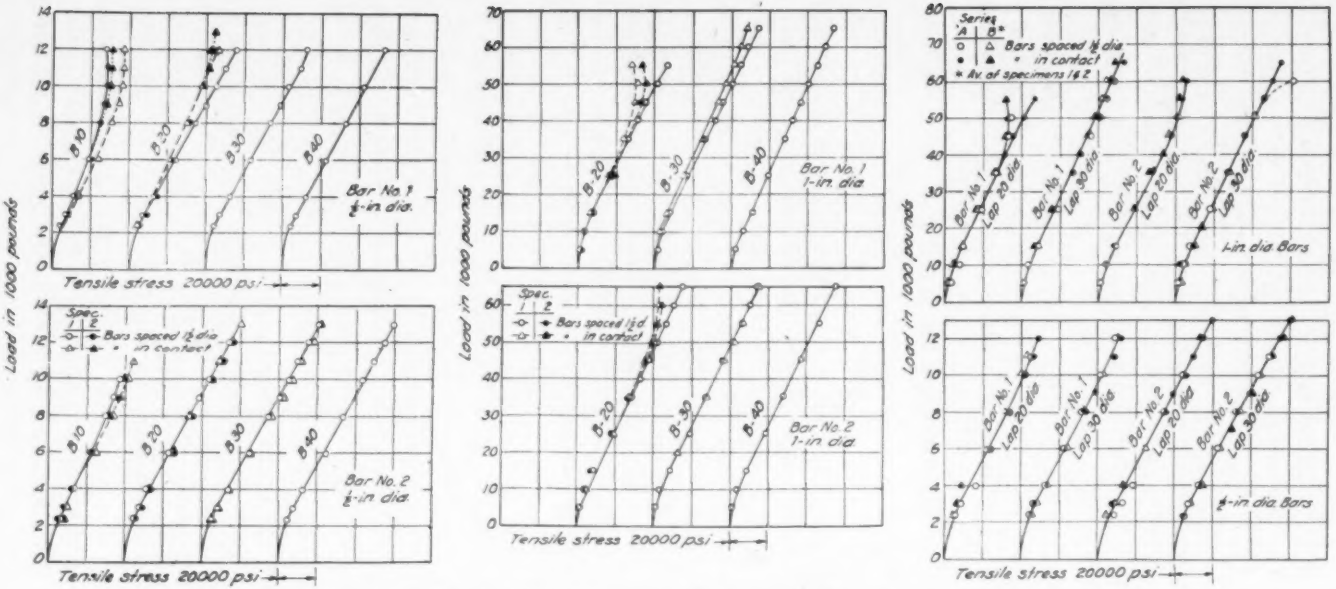


Fig. 25 (left)—Load-stress relation for lapped bars outside limits of splice—beams of series B. ½ in. bars.

Fig. 26 (center)—Load-stress relation for lapped bars outside limits of splice—beams of series B. 1 in. bars.

Fig. 27 (right)—Comparison between load-stress diagrams of series A and series B. Stress outside limits of splice.

Stress values are the product of the modulus of elasticity and observed strains.

TABLE 5—AVERAGE BOND STRESSES DEVELOPED BY LAPPED BARS JUST BEFORE FAILURE

Tests of Series B

Specimen	No.	Length of embedment	Bar No. 2		Bar No. 1	
			Tensile stress	Bond stress	Tensile stress	Bond stress
		in.	psi.	psi.	psi.	psi.
			1/2-in. bars			
B40	1	22¾	*60,000	325	*52,000	300
B30	1	17¾	*60,000	420	*55,000	385
B30x	1		*60,000	420	———	—
B20	1	12¾	*60,000	580	*55,000	535
	2		*60,000	580	43,500	425
B20x	1	12¾	*60,000	580	47,000	460
	2		*60,000	580	43,500	425
B10	1	7¾	39,000	625	28,500	455
	2		45,000	720	28,500	455
B10x	1	7¾	45,000	720	36,800	590
	2		40,500	650	30,600	490
			1-in. bars			
B40	1	43	*60,000	350	*53,000	310
B30	1	33	*60,000	455	*53,000	405
B30x	1	33	*60,000	455	**49,000	370
B20	1	23	*60,000	655	45,000	490
	2		43,500	475	30,000	330
B20x	1	23	43,500	475	30,000	330
	2		45,000	490	33,800	370

*Tension failures.
**Tension failures in continuous bar.

the performance of the lapped bars, which were in contact with each other again compare favorably, with those spaced 1½ diameters apart.

11. Effect of gage holes on the average bond stress

Undoubtedly the exposure of the bar at each gage hole affected the bond strength in the immediate vicinity of the hole. However, there was some evidence to indicate that the average bond strength of the entire embedded length of the bar was not significantly affected by these holes. Within the region of the splices in beams of series A, the holes in the concrete exposed approximately 4 percent and 7 percent of the surface area of the 1-in. and ½-in. bars, respectively. Fig. 27 shows load-stress diagrams for the 20-and 30-diameter lapped bars at a point 2½ in. beyond the ends of the splices. Comparison of these diagrams reveals

little, if any, difference in the data from tests of series A and series B at this point. The data from series B, represented by solid symbols in the diagrams, are the average values obtained from duplicate tests, whereas the open symbols represent but a single test. Ordinarily, a comparison of the maximum loads between corresponding beams, at least for the shorter splices, would indicate the effect of exposing the reinforcement in this manner. However, ultimate failure of most of the beams in series A was due to yielding of the reinforcement.

DISCUSSION OF RESULTS

12. General discussion

It is recognized that many of the data presented are based on single tests, from which sound conclusions may not always be drawn. It is significant, however, that, except where observations of bar slip were concerned, the data were consistent from specimen to specimen. For example, comparisons of bond stress and its manner of distribution were made between the two types of splices and, although each comparison was based on a single test, the six different specimens exhibited consistently similar characteristics. Differences in the behavior of the two types of bars were likewise consistent.

Curves showing the distribution of stress along the lapped bars should, to some extent, be considered as qualitative. They indicate the general manner of stress distribution for each length of splice. Average values from two or more tests would perhaps differ from those shown.

It should be noted also that the splices which were tested were single bar splices flanked by two continuous bars. Different results might have been obtained had the specimens contained multiple lap splices without the accompanying continuous bars. Previous unpublished tests of beams constructed of Haydite concrete and containing multiple splices only, indicated that a greater cover than normally provided around the outermost bars of the beam was necessary if they were to assume their share of the total stress. Lapping of all bars at a section of a flexural member is, of course, not to be recommended in practice. The presence of continuous bars prevents the sudden and violent failure that may occur when all bars are lapped.

SUMMARY OF TEST RESULTS

13. General summary

The following briefly summarizes the results of the tests:

(1) The bars lapped 30-bar diameters or more developed, at the limits of the splice, the yield strength of the steel, a value which in some instances was as much as 60,000 psi.

(2) There was little difference observed in the behavior of splices containing bars spaced 1½ bar diameters clear or with the lapped bars in contact. It should be noted that, as far as bond resistance is concerned, the bars used in these tests contained a type of lug pattern probably superior to many commercial bars.

(3) Bond stress was the greatest near the free end of the lapped bar. The maximum values occurred at approximately the same tensile stress at the loaded end of the bar, regardless of the length of lap. A possible exception to this was the 20-bar diameter splice.

(4) With two exceptions, the magnitude and distribution of the slip along the lapped bars was similar for a given type and size of bar, regardless of the length of lap.

(5) The general behavior of the ½-in. bars was similar to the 1-in. bars, in so far as bond and tensile stresses were concerned. Differences in bar slip are noted in item 8.

(6) There was some evidence to indicate that the gage holes in the concrete of the beams did not seriously affect the strength of the splice.

14. Comparison of the two types of bars

(7) Bar No. 2, which had a considerably greater bearing area of the lug per inch length of bar, not only picked up stress at its free end in the splice more rapidly but developed greater maximum bond strengths than bar No. 1.

(8) The 1-in. diameter No. 1 bar exhibited a considerably greater slip for a given bond stress than any of the other bars. The differences between the two types of bar of the ½-in. size were minor. Both showed slightly less end slip than the 1-in. No. 2 bar for a given bond stress at the free end of the bar.

[To facilitate selective distribution, **separate prints** of this title (42-3) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Mar. 1, 1946.]

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Tests of Prestressed Concrete Pipes Containing A Steel Cylinder*

By CULBERTSON W. ROSS†

SYNOPSIS

Tests were made on prestressed reinforced concrete pipes of a type containing a steel cylinder. Data were obtained by tests under hydrostatic pressure, in crushing and in bending. The mechanical properties of the several parts, and the strain changes of the pipes under load are reported.

INTRODUCTION

A type of prestressed reinforced concrete pipe containing a steel cylinder was subjected to tests by the National Bureau of Standards. The pipes were furnished by, and the tests were made in cooperation with, the Water Division, Engineer Department of the District of Columbia. This report gives the results of hydrostatic pressure, bending, and crushing tests. The strain changes during the tests and the properties of the component parts of the pipes are also given. These will enable an engineer to judge the suitability of similar pipes for particular applications.

DESCRIPTION OF THE PIPES

The pipes consisted of a concrete-lined 16-gage welded sheet steel cylinder upon which was a wrapping of steel wire with a mortar covering, as indicated in Fig. 1. The dimensions of the pipes and their several parts are given in Table 1.

The lining of the 20-in. pipes was of crushed stone concrete and that of the 30-in. pipes was of gravel concrete. The linings were cast by spinning the steel tubes as the concrete was placed. The spinning expelled much

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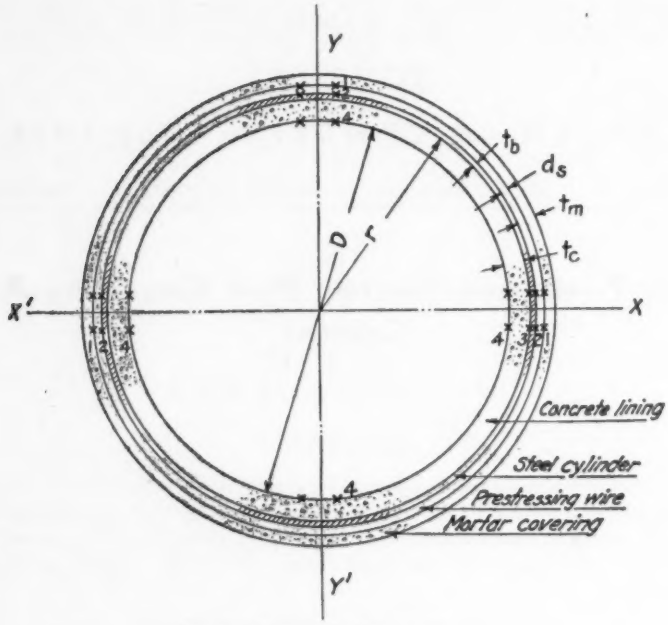

Fig. 1—Diagram of the pipe
The small "x" marks indicate the location of the strain gage stations.

of the water and compacted the concrete. A layer of material was formed at the inner surface of the lining which consisted chiefly of cement, whose thickness in the 20-in. pipes was about 1/16 in. and in the 30-in. pipes about 1/4 in. and in the latter the layer contained a network of shrinkage cracks.

The cylinder was wound with wire under high tensile stress after the lining had hardened. The wire was bonded to the pipe and protected from corrosion by the covering of mortar.

The device for sealing the pipes at the joints consisted of two steel rings of heavier gage than the cylinder and of different diameters, welded to the ends of the cylinder. A rubber ring was placed in a groove in the smaller ring of two adjacent pipes and compressed between the outer surface of one ring and the inner surface of the other as the pipes were drawn together*.

SCOPE OF TESTS

Hydrostatic pressure tests were made on one 20-in. and two 30-in. pipes, beam tests on one 20-in. and one 30-in. pipe, and crushing tests on

*More complete descriptions of the pipes and joints as well as test data on the mechanical properties of the pipes are given in the "Report on Lock Joint Prestressed Concrete Cylinder Pipe," Underwriters Laboratory, Inc., Nov. 24, 1944.

TABLE 1—DIMENSIONS OF PIPE AND MECHANICAL PROPERTIES OF MATERIALS

Dimension or property	Symbol	20-in. Pipe	30-in. Pipe
Internal diameter of pipe, in.	D	20	30
Length, ft.		16	16
Thickness of concrete lining, in.	t_c	1.22	1.85
Thickness of steel cylinder, in.	t_b	0.06	0.06
Thickness of mortar cover, in.	t_m	0.75	0.75
Diameter of wire, in.	d_s	0.19	0.19
Spacing of wire, in.		1.00	0.80
Cross-sectional area of wire per inch length of pipe, one wall, sq. in.	A_s	0.0284	0.0354
Cross-sectional area of cylinder per inch length of pipe, one wall, sq. in.	A_b	0.06	0.06
Cross-sectional area of lining per inch length of pipe, one wall, sq. in.	A_c	1.22	1.85
Radius of cylinder, in.	r	11.22	16.85
Yield point of wire (offset 0.2 percent), psi.	Y_s	170,000	170,000
Yield point of cylinder (offset 0.2 percent), psi.	Y_b	33,000	33,000
Tensile strength of wire, psi.	T_s	208,000	208,000
Tensile strength of cylinder, psi.	T_b	52,000	52,000
Elongation of wire (in 8-in. length), percent		4	4
Elongation of cylinder (in 2-in. length), percent		30	30
Modulus of elasticity of wire, psi.	E_s	27,500,000	27,500,000
Modulus of elasticity of cylinder, psi.	E_b	29,100,000	29,100,000
Compressive strength of concrete of lining (3.7 by 2.1 by 1.8-in. prisms), psi.		—	7,600
Modulus of elasticity of concrete of lining, psi.	E_c	—	4,200,000
Ratio E_b/E_c	n	—	7
Prestrain in wire	—	—	0.00237
Prestrain in cylinder	—	—	0.00020
Prestress in wire, psi.	f'_s	—	+65,000
Prestress in cylinder, psi.	f'_b	—	−5,800
Prestress in concrete, psi.	f'_c	—	−1,050
Maximum pressure sustained by pipe, psi.	P_m	740	(570 (600
Working pressure (original design), psi.	P_w	192	192
Crushing strength, lb./ft.	—	12,300	13,000
Beam strength, 14-ft. span, lb.	—	55,000	140,000

two 4-ft. lengths of the 20-in. and two of the 30-in. pipes. The two crushing test specimens of the 20-in. pipe were taken from the less stressed portions of the pipe used in the beam test. Those for the crushing test of the 30-in. pipe were cut from an untested pipe and the remaining portion of this pipe was the source of material for the tests of mechanical properties and prestress.

For the hydrostatic test, each end of the pipe was closed by a bulkhead which was sealed by the fittings used for connecting pipes in water mains. Each bulkhead consisted of a steel ring, corresponding to one of those at the end of the pipe, and a steel plate welded to one edge of the ring. The bulkheads and the rubber rings were forced onto the ends of the pipe by the plates of the testing apparatus. The prepared pipe was first filled completely with water and then an additional amount was pumped in to

build up the pressure. The pressure increased rapidly until the pipe started to expand because of yielding of the reinforcing, after which it increased slowly. About two hours of pumping were required for the pressure to reach its maximum value, of which one and a half hours were required for the last 100 psi. increase. Strain changes in the wire were measured at X_1 and X'_1 (Fig. 1) at each 50 psi increment of pressure. The pressure was held constant while the readings were being made for a period of about 5 min. at each increment. A 2-in. Whittemore strain gage was used for the measurements of the 20-in. pipe and a 5-in. Whittemore gage for those on the 30-in. pipes.

For the beam tests, the pipes were supported and the load applied through V-blocks having interior angles of 120 deg. The blocks were cut from timbers of nominal size 12 by 12 in., giving a bearing length along the side of the pipe of about 11½ in. Timber struts, size 4 by 4 in., were placed vertically in the pipes at the center and at the supports to help prevent local crushing of the pipe. The pipes were loaded at the center of a 14-ft. span, measured center to center of the blocks. The deflection, measured with a surveyor's level, was observed for the 20-in. pipe only.

In the crushing tests, 4-ft. lengths of pipe were tested by the three-edge bearing method, described in A.S.T.M. Specification C 14-41. Measurements were made of the decrease in the vertical diameter, of the change of the strain at the inner surface of the lining at the top and bottom, and at the sides, and of the change of the strain in the tube and wire at the top and sides. A 5-in. Whittemore strain gage was used for the measurements made on the 30-in. pipe specimens and the outside of the 20-in. pipe specimens. The measurements of the lining of the 20-in. specimens were made with a 2-in. Whittemore gage.

The tensile strength, the yield strength, the modulus of elasticity and the elongation of the wire and of specimens of the steel cylinder were measured by the methods of test of A.S.T.M. E 8-42.

Two prisms approximately 3.7 by 2.1 by 1.8 in. and two approximately 2.1 by 2.1 by 1.8 in. were cut from the lining of a 30-in. pipe. The prisms were tested in compression and the modulus of elasticity measured with the pressure applied to the 2.1 by 1.8-in. faces. The prisms were dry from several weeks exposure to the air when tested.

The prestrain in the lining, cylinder and wire of the 30-in. pipe, from which the crushing strength specimens had been cut, was found by measuring the change in length of the parts with a 5-in. Whittemore strain gage when the prestress was released by cutting out a 12- by 12-in. section. The lining of the pipe was air dry at the time the tests were made.

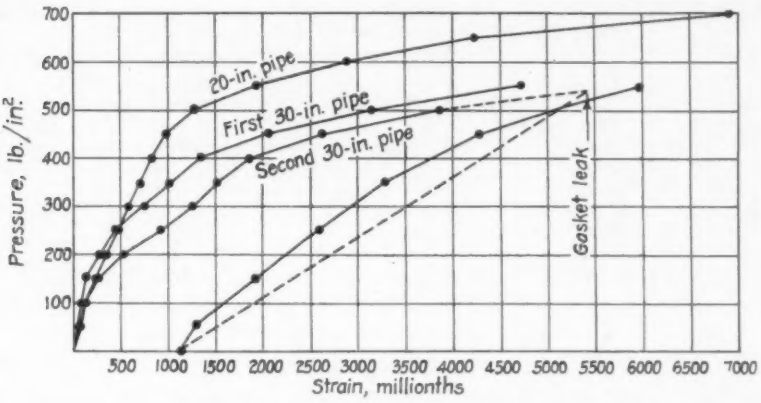

Fig. 2—Increase of the strain in the wire during the hydrostatic tests of three pipe.

TESTS AND RESULTS

Hydrostatic pressure tests

The maximum pressure sustained by the 20-in. pipe was 740 psi.

The maximum pressures sustained by the two 30-in. pipes were 570 and 600 psi. A leak appeared along a longitudinal seam of the first pipe at about 200 psi causing the mortar to be damp for a length of about 10 ft. The leak stopped as the test was continued and the damp spots disappeared completely before the end of the test. In the test of the second 30-in. pipe, the pressure was released at a pressure of 540 psi because of a gasket leak. The leak was repaired and the pressure again increased until the pipe failed.

As the pressure increased, cracks appeared in the mortar covering of the three pipes at about 200 psi and became rather numerous and large as the maximum pressure was approached. At the upper limit of the pressure one or more of the wires broke, neighboring turns loosened, and the cylinder was expanded outward in a ring around the pipe which was from 8 to 12 in. wide and about 1 in. high. The cylinders did not leak in spite of the large deformation of the expanded portion and still held a pressure substantially higher than the designed pressure (192 psi).

Strain changes in the wire were measured at X and X′ (Fig. 1) during the tests and are shown in Fig. 2.

Beam tests

Cracks became visible in the outer mortar covering of the 20-in. pipe at a load of about 20,000 lb. and in the 30-in. pipe at about 80,000 lb. The maximum load supported by the 20-in. pipe was 55,000 lb. The failure occurred by the buckling of the cylinder and the spalling of the lining at

TABLE 2—DEFLECTION OF THE CENTER OF THE 20-IN. PIPE RELATIVE TO THE ENDS OF THE SPAN DURING THE BEAM TEST

Load	Deflection
Lb.	In.
5,000	0.02
10,000	.04
20,000	.12
30,000	.20
40,000	.36
50,000	1.14
54,000	1.70

the top of the pipe. The maximum load supported by the 30-in. pipe was 140,000 lb. Failure occurred by the cylinder tearing at the bottom.

The deflections of the 20-in. pipe under load are given in Table 2.

Crushing tests

The data from the measurements of the strains at various points in the pipes are shown in Fig. 3 and 4 for one specimen of 20-in. and one of 30-in. pipe. The locations of the points at which strains were measured are shown in Fig. 1. The strains given for the inside of the lining at the top and bottom include the widths of the cracks at those points. After the lining cracked, the widths of the cracks in inches are approximately twice the value of the strain for the 20-in. pipe because of the 2-in. gage length and five times the value shown for the 30-in. pipe because of the 5-in. gage length (Fig. 3 and 4, Y_4 and Y'_4).

The maximum loads sustained by the 20-in. pipe specimens were 49,800 and 48,500 lb. or an average of 12,300 lb. per ft. of length. The maximum loads for the 30-in. pipe specimens were 51,500 and 52,000 lb., or an average of 13,000 lb. per ft. of length. The specimens failed by the crushing of the concrete of the lining under the top loading block.

Tests of materials of the pipes

The tensile strength, the yield strength, the modulus of elasticity and the elongation of the wire and of specimens of the steel cylinder are given in Table 1.

The average compressive strength of the 3.7- by 2.1- by 1.8-in. was 7,600 psi. and of the 2.1- by 2.1- by 1.8-in. prisms was 10,100 psi.* The modulus of elasticity of the taller prisms was found to be 4,200,000 psi.

The prestrain in the lining at the inner surface was found to be 0.00046 (point X_4, Fig. 1), at the outer surface, adjacent to the cylinder (point X_3)

*The 28-day strength of 6- by 12-in. cylinders, made from concrete of the lining before it was placed in the pipe, was reported by the District of Columbia to have ranged from 3,000 to 5,000 psi, with an average of about 4,000 psi.

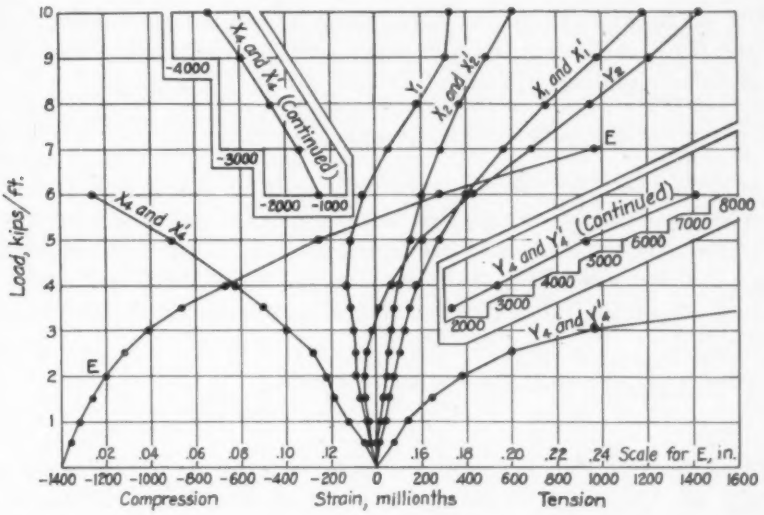

Fig. 3—Change of the strain at various points of the 20 in. pipe during the crushing test.

Y_1 wire at the top.
Y_2 cylinder at the top.
Y_4&Y'_4 inner surface of lining at top and bottom.
X_4&X'_4 inner surface of lining at sides.
X_1&X'_1 wire at the sides.
X_2&X'_2 cylinder at the sides.
E decrease in vertical diameter.

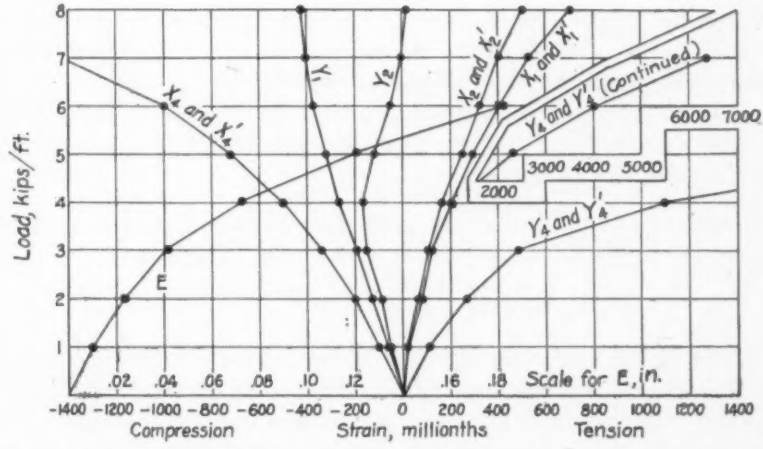

Fig. 4—Change of the strain at various points of the 30-in. pipe during the crushing tests.

Y_1 wire at the top.
Y_2 cylinder at the top.
Y_4&Y'_4 inner surface of lining at top and bottom.
X_4&X'_4 inner surface of lining at sides.
X_1&X'_1 wire at the sides.
X_2&X'_2 cylinder at the sides.
E decrease in vertical diameter.

0.00010, in the cylinder (point X_2) 0.00020, and in the wire (point X_1) to be 0.00237. The stress in the wire when it was wound was reported by the District of Columbia to have been 86,000 psi.

DISCUSSION

The design of this type of prestressed concrete pipe is similar to that described by Crepps*, except that the effect of the cylinder must also be considered. The spacing and size of the wire and the prestresses may be chosen so that there will be no circumferential stress in the concrete at the working pressure. Under these conditions, the working pressure is given by (Table 1):

$$P_w = \frac{-n f'_c (A_s + A_b + A_c/n)}{r}$$

It was reported that the pipes tested had been designed for a working pressure of 192 psi (112 psi operating head plus 80 psi allowance for water hammer head), based on slightly different values for the constants than those given in Table 1. Pressure in excess of this value existing for short periods should not injure the pipe, as any small cracks in the lining would close again when the excessive pressure is removed. However, pressures or loads which cause the strain in either the wire or the cylinder to exceed their elastic limits might cause permanent cracks in the concrete. An inspection of Fig. 2 indicates that the yield point of either the wire or cylinder was reached at a pressure of about 500 psi in the 20-in. pipe and at about 400 psi in the 30-in. pipes. The values of the yield point and the prestrain of the wire and cylinder of the 30-in. pipe indicate that the change in slope of the pressure-strain curves at about 400 psi. pressure was caused by the yielding of the cylinder. A less well marked change in slope of the pressure-strain curve for the 30-in. pipes may be noted between 150 and 200 psi, which should indicate approximately the pressure at which the concrete cracked.

When there is no pressure acting on the pipe, the prestress in the wire balances the prestress in the cylinder and lining. Therefore

$$f'_s A_s + f'_b A_b + f'_c A_c = O$$

The stress in the wire when the pipe is wound must be higher than the desired final prestress because of the allowance which should be made for shrinkage, plastic flow, and the elastic deformation of the cylinder and lining. For the 30-in. pipes, the winding stress of 86,000 psi may be compared with the stress computed from the measured prestrain of 65,000 psi.

*Wire-Wound Prestressed Concrete Pressure Pipe, Ray B. Crepps, ACI JOURNAL, June 1943; *Proceedings*, V. 39, p. 545.

However, the latter prestress value would probably be higher if the lining were wet.

The pressure which will cause these pipes to burst can be estimated approximately from the properties of the wire and material of the cylinder from the equation

$$P_m = \frac{T_s A_s + Y_b A_b}{r}$$

This gives P_m as 705 psi for the 20-in. pipe and 555 psi for the 30-in. pipes. These pressures were exceeded in the tests.

While the tests were made primarily to investigate the properties of the pipes, it may be noted that the joints also successfully withstood the maximum applied pressures.

[To facilitate selective distribution, **separate prints** of this title (42-4) are currently available from ACI at 50 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Mar. 1, 1945.]

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Field Use of Cement Containing Vinsol Resin*

By CHARLES E. WUERPEL†
Member American Concrete Institute

SYNOPSIS

The results obtained from 22,398 test specimens manufactured in connection with extensive construction, principally during the period 1941-1944, are presented together with a discussion of the experience with handling concrete containing over 2,000,000 barrels of Vinsol resin cement during this period in 168 structures. Comparisons are drawn between concrete made with plain cement and with cement interground with Vinsol resin as they affect the compressive and flexural strength, the unit weight and the mixing, placing and finishing operations.

INTRODUCTION

The use of portland cement containing Vinsol resin, as an interground addition, in field construction was inaugurated within the Department in April 1941 in the concrete walls of a flood control structure at Corning, N. Y. This type of cement was used upon the recommendation of this laboratory as an experiment designed to reduce to a practicable minimum the excessive bleeding and resultant sand-streaking which occurred in the vertical sections of the walls when plain cement was used. The experiment was a success and additional cement of this type was used in a flood wall at Binghamton, N. Y., commencing in May 1941.

The highly salutary effect of Vinsol resin, as an interground addition to portland cement, upon the durability of concrete obtained by this laboratory in tests at Treat Island, Maine (1*), together with information received of similarly desirable properties exhibited in experimental highway projects (2, 3, 4, 5, 6, 7, 8, 9) and the practical improvements of workability and bleeding in the two flood protection structures led to

*Received by the Institute Jan. 8, 1945.
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*Figures in parentheses refer to bibliography appended to this paper.

TABLE 1—CEMENT COMPANIES AND MILL LOCATION FROM WHICH VINSOL RESIN CEMENT HAS BEEN SHIPPED TO PROJECTS

Cement Company	Mill Location
Allentown Portland Cement Co.	Evansville, Penn.
Alpha Portland Cement Co.	Cementon, N. Y.
Alpha Portland Cement Co.	Jamesville, N. Y.
Coplay Cement Co.	Coplay, Penn.
Federal Portland Cement Co.	Buffalo, N. Y.
Giant Portland Cement Co.	Egypt, Penn.
Glens Falls Portland Cement Co.	Glens Falls, N. Y.
Hercules Portland Cement Co.	Stockertown, Penn.
Huron Portland Cement Co.	Oswego, New York
Keystone Portland Cement Co.	Bath, Penn.
Lawrence Portland Cement Co.	Northampton, Penn.
Lawrence Portland Cement Co.	Thomaston, Maine
Lehigh Portland Cement Co.	Alsen, New York
Lehigh Portland Cement Co.	Buffalo, New York
Lehigh Portland Cement Co.	Fogelsville, Penn.
Lehigh Portland Cement Co.	Fordwick, Virginia
Lehigh Portland Cement Co.	Ormrod, Penn.
Lone Star Cement Corp.	Hudson, New York
West Penn. Cement Co.	Butler, Penn.
National Portland Cement Co.	Brodhead, Penn.
Nazareth Cement Co.	Nazareth, Penn.
North American Cement Corp.	Alsen, New York
North American Cement Corp.	Howe Caves, New York
Penn-Dixie Cement Corp.	Bath, Penn.
Penn-Dixie Cement Corp.	Clinchfield, Ga.
Penn-Dixie Cement Corp.	Nazareth, Penn.
Penn-Dixie Cement Corp.	Portland Point, N. Y.
Universal Atlas Cement Co.	Hudson, New York
Universal Atlas Cement Co.	Leeds, Alabama
Universal Atlas Cement Co.	Northampton, Penn.
Whitehall Cement Co.	Northampton, Penn.

cement of this type being specified for the runways, roads and buildings of the Rome Air Depot, Rome, N. Y. in September 1941. The 435,706 barrels of SS-C-206a cement containing Vinsol resin (a similar specification to that later accepted by the A.S.T.M. as C 175-42T) used in the various elements in this project up to date was produced by 13 mills operated by 9 companies and constituted the first use of this type of cement in major amounts in this country.

The excellent results obtained during the construction of the three projects discussed above led to a more general requirement for this type of cement in construction by the Department in the northeastern portion of the United States. Up to August 24, 1944, 2,037,851 bbl. of cement containing Vinsol resin have been tested by this laboratory at 31 mills for use on 168 separate projects. The mills which have manufactured the cement are listed in Table 1. The types and number of structures in which the cement has been used are listed in Table 2. It is the purpose of this paper to present the results obtained from tests on the numerous specimens prepared in the field and a discussion of the experience gained with this type of cement in concrete construction. It is proposed to

TABLE 2—TYPES AND NUMBER OF STRUCTURES IN WHICH CEMENT CONTAINING VINSOL RESIN HAS BEEN USED

Type of Structure	Number
Pavements (Access roads and airport runways)	70
Floors	
Hangar	7
Warehouse	39
Building	76
Walls	
Flood	8
Warehouse	38
Building	76
Retaining	11
Sea	1
Target Butts	8
Foundations	
Gun Blocks (Major)	4
Footings	100
Total Number	438

present the results of the extensive laboratory investigations conducted concurrently in a separate paper.

Cement containing Vinsol resin has been used in field construction only during the period of the national emergency when demands for speed were great, the supply of skilled inspectors and technicians was absorbed largely into the armed forces and equipment was frequently below standard. Accordingly, the control of concrete mixtures and the accuracy with which records were kept was imperfect. Concrete made with plain cement was not subject to the effect of this lowered efficiency to the degree that prevailed for concrete containing Vinsol resin; use of plain cement was not new and was reasonably well standardized. Use of cement containing Vinsol resin was new and its effect on mixture design, the behavior of the plastic mixture and on the hardened concrete was not well understood. Frequently, cement containing Vinsol resin was placed in mixtures which had been designed for plain cement. In most of these cases, some adjustment of the mixture proportions was made because of the radical increase in consistency or slump, but the nature of the adjustment was not always recorded and may have been a simple reduction in water content, or cement content.

Except for the inadequacy of field records, the irregularities of design and their effects on the hardened concrete might have been analyzed with greater benefit from the experience. However, without such records, it is possible only to present the test results as obtained and use limited instances, where the causes of abnormalities were known, to guess the cause for other similar and sometimes dissimilar abnormalities.

The importance of the effect of the amount of entrained air on the strength of the concrete was not recognized during the early use of Vinsol resin. Accordingly, no routine efforts were made or considered necessary

to determine or regulate the quantity of entrained air by field tests on the plastic concrete. The necessity for measures of control beyond those provided by the cement specification (quantitative limit on Vinsol resin content) was demonstrated when very low strengths developed in the concrete placed in the aprons and turn-arounds at Grenier Field, Manchester, N. H., during the autumn of 1942. Density tests of cores extracted from the work early in 1943 indicated an air content in some locations as high as 15 per cent and a general air content of approximately 11 per cent.

A generally similar experience occurred in the autumn of 1942 in the development of abnormally high air contents and correspondingly low compressive and flexural strengths in sections of the runways at Trumbull Airfield, Groton, Conn., in which Vinsol resin was an interground addition to the cement.

These two major and three other minor occurrences of an abnormal air content, focused the attention of the laboratory on the importance of field tests for measurement and control of the air content. The fact that the cements used on these and all other projects were tested by this laboratory and found to contain Vinsol resin in amounts within the limits specified (0.025 – 0.045 per cent by weight of cement) and to develop acceptable strength in mortar tests, indicated strongly that limiting the amount of interground Vinsol resin was inadequate for desired results. Laboratory investigations were started to determine why certain cements caused excessive air-entrainment while others with a similar or lesser amount of Vinsol resin caused normal air entrainment. The nature of these investigations, and the efforts made to eliminate this variable and the results obtained were presented in the A.S.T.M. Bulletin for Oct. 1944 (10) and in the 1944 Annual Report of Committee C-1 of the A.S.T.M. (11). Amplification of the laboratory work will be presented in a later paper.

With the conditions of imperfect records in mind as a limitation of accuracy, the results of tests made on field concrete are presented.

STRENGTH

Field control cylinders*

Test cylinders representing concrete placed on projects constructed by the Districts of the North Atlantic Division and certain districts of other Divisions, are forwarded to this laboratory for test. Between January 1937 and March 1944, 37,910 such cylinders were tested, of which 16,819 were tested at the age of 28 days. Information regarding each cylinder is required to be furnished by the project to the laboratory. On the basis of this information, the data have been classified for analysis in the present

*The data referred to are taken from a report entitled "Compressive Strength at 28-Days Age of Job-Made Concrete Cylinders Containing Treated and Untreated Portland Cement" under the date April 1944 which was distributed in September 1944 within the Department by this office (12).

TABLE 3—NUMBERS OF SPECIMENS TESTED AT 28-DAYS

Classified by districts in which made, type of cement used and period in which tested

Period	1937-1943		1943-1944		1937-1944		Total
District	Plain	V. R.*	Plain	V. R.	Plain	V. R.	Total
Baltimore	1601	....	406	....	2007	....	2007
Boston	1383	69	599	66	1982	135	2117
New York	1997	20	583	645	2580	665	3245
Philadelphia	318	....	577	158	895	158	1053
Providence	3184	93	404	33	3588	126	3714
Syracuse	3307	938	138	300	3445	1238	4683
Total	11790	1120	2707	1202	14497	2322	16819
Total	12910		3909		16819		....

*Use commenced April 1941.

paper. For the purpose of compressive-strength analysis, classification groups have been made on the following bases:

(1) Reported water-cement ratio,
(2) Reported type of cement used (plain or containing Vinsol resin), and
(3) Period in which tested (before or after March 1943).

As additional information, a classification of the specimens as to source, number and type is given by districts in Table 3.

(a) *Water-cement ratio.* The number of specimens of concrete made with plain cement and cement containing Vinsol resin reported for the various water-cement ratios is given in Table 4 and indicated graphically in Fig. 1. The largest groups of specimens made with plain cement and with cement containing Vinsol resin are found to have reported water-cement ratios of 6.0 and 5.0 gal. per bag, respectively. This evidence reported by the field substantiates laboratory experience which has indicated that concrete of similar cement factor and degree of workability can be obtained with cement containing Vinsol resin using a water-cement ratio approximately one gallon per bag less than would be needed with plain cement.

(b) *Compressive strength.* A tabulation of average compressive strengths of cylinders classified by water-cement ratio, type of cement, and period in which tested, is given in Table 5. The relationship between compressive strength and water-cement ratio for each type of cement, in each period, and for the data as a whole, are indicated by the graphs in Fig. 2. Fig. 2 (D) shows the comparability of the compressive strength-water-cement ratio relationship for VR treated and plain-cement concrete when proper cognizance is taken of the effect of VR in increasing workability as mentioned in (a) above. Also shown on these graphs is the curve indicating the values assumed by this laboratory for the compressive strengths normally to be anticipated at 28-days for concrete made with plain

TABLE 4—NUMBERS OF SPECIMENS TESTED AT 28-DAYS

Classified by Reported Water-Cement Ratio, and Type of Cement Used

Reported Water-Cement Ratio	Plain Cement	Cement with V. R.
gal. per bag		
3.0	22	...
3.5	3	14
4.0	200	101
4.5	278	105
4.75	....	182
5.0	903	853
5.25	20	57
5.5	3943	648
5.75	93	..
6.0	7373	277
6.25	7	4
6.5	1136	35
6.75	195	3
7.0	203	41
7.5	86	1
8.0	31	....
8.5	3	....
9.5	1	....
11.0	....	1
Total	14497	2322

cement conforming to Federal Specification SS-C-206a and the water-cement ratios shown.

Notes to Table 5 indicate the selectivity exercised in the plotting of the data in Fig. 2. Five other values (Note 3, Table 5) were modified or omitted prior to the plotting of Fig. 2. In four of these cases evidence cast serious doubt on the accuracy of the reported water-cement ratio for cylinders from one particular project or district. In these cases, the doubtful value was omitted. In the fifth case, a peculiarly faulty distribution of data occurred which, due to the domination of successive weighted averages by data from different sources, gave an anomalous relation. These data are tabulated:

	Plain Cement — 1943-44			
	W/C = 7.5		*W/C = 8.0*	
	No. of Tests	*psi.*	*psi.*	*No. of Tests*
Baltimore	3	4360	4135	2
New York	20	2570	——	—
Providence	6	3285	3275	16
Weighted averages	——	2905	3370(a)	—

(a) Omitted value.

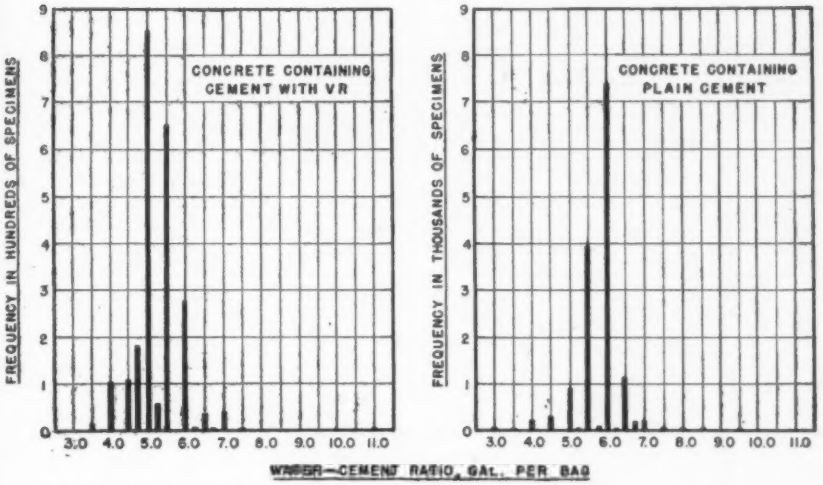
CONCRETE CONTAINING CEMENT WITH VR
CONCRETE CONTAINING PLAIN CEMENT
FREQUENCY IN HUNDREDS OF SPECIMENS
FREQUENCY IN THOUSANDS OF SPECIMENS
3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0 11.0
WATER—CEMENT RATIO, GAL. PER BAG

Fig. 1—Relation of frequency of specimens to water-cement ratio of concrete containing cement with and without vinsol resin.

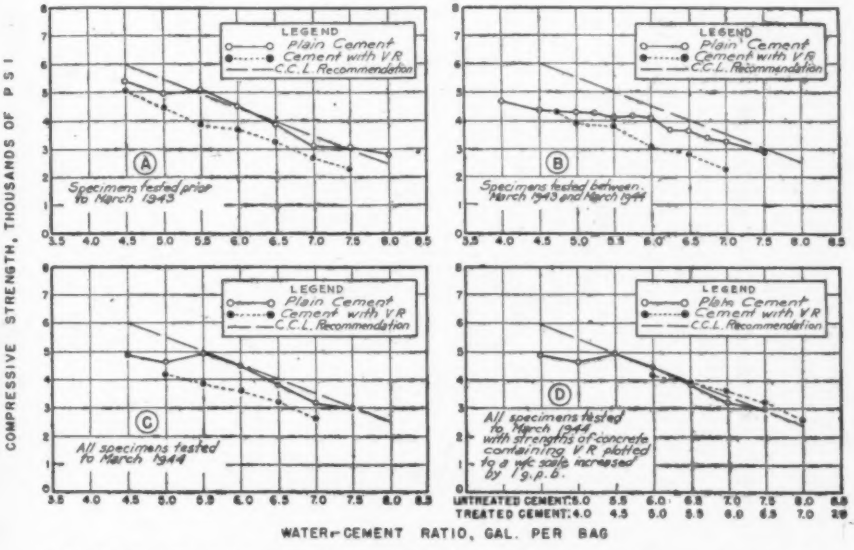
LEGEND
Plain Cement
Cement with V.R
C.C.L. Recommendation
A
Specimens tested prior to March 1943
B
Specimens tested between March 1943 and March 1944
C
All specimens tested to March 1944
D
All specimens tested to March 1944 with strengths of concrete containing V.R plotted to a w/c ratio increased by 1 g.p.b.
COMPRESSIVE STRENGTH, THOUSANDS OF P S I
UNTREATED CEMENT: 3.5 4.0 4.5 5.0 5.5 6.0 6.5 7.0 7.5 8.0 8.5
TREATED CEMENT: 4.0 4.5 5.0 5.5 6.0 6.5 7.0 7.5 8.0 8.5 9.0
WATER-CEMENT RATIO, GAL. PER BAG

Fig. 2—Relation of compressive strength of job cylinders to water-cement ratio of concrete containing cement with and without vinsol resin (age: 28 days)

It is probable that all tabulated values are incorrect with respect to the reported water-cement ratio, however, the general average value of 2905 psi, for a water-cement ratio of 7.5 gallons per bag is considered to be reasonable.

TABLE 5—AVERAGE COMPRESSIVE STRENGTHS OF SPECIMENS TESTED AT 28 DAYS

Classified by reported water-cement ratio, type of cement, and period in which tested.

Period: 1937 to March 1943

Plain Cement							Cement with V. R.						
Report. W/C Ratio	No. of Spec.	Comp. Str. psi	No. Discarded	Remaining Spec.	Corr. Comp. Str. psi	Notes	Reprt. W/C Ratio	No. of Spec.	Comp. Str. psi	No. Discarded	Remaining Spec.	Corr. Comp. Str. psi	Notes
3.0	1	7000	1	0	....	(*)							
3.5	3	3750	3	0	....	(*)							
4.0	62	4385	62	0	....	(†)							
4.5	150	5350	0	150	5350	....	4.5	1	5090	0	1	5090	....
5.0	512	4590	128	384	4960	(‡)	5.0	410	4495	0	410	4495	....
5.5	3405	5065	0	3405	5065	....	5.5	437	3920	0	437	3920	....
6.0	6594	4540	0	6594	4540	....	6.0	219	3760	0	219	3760	....
6.5	869	3875	0	869	3875	....	6.5	15	3265	0	15	3265	....
7.0	122	3130	0	122	3130	....	7.0	37	2680	0	37	2680	....
7.5	57	3025	0	57	3025	....	7.5	1	2250	0	1	2250	....
8.0	13	2790	0	13	2790	....							
8.5	1	3435	1	....	....	(*)							
9.5	1	910	1	....	....	(*)							
Total	11790	....	196	11594	....	....	Total	1120	.....	....	1120	....	....

*Insignificant representation.
†Inaccurate W/C data, water in aggregate ignored.
‡Special Cases—see text.

Table 5 continued on opposite page

TABLE 5—Continued from opposite page

Period: March 1943 to March 1944

Plain Cement							Cement with V. R.						
Report. W/C Ratio	No. of Spec.	Comp. Str. psi	No. Discarded	Remaining Spec.	Corr. Comp. Str. psi	Notes	Report. W/C Ratio	No. of Spec.	Comp. Str. psi	No. Discarded	Remaining Spec.	Corr. Comp. Str. psi	Notes
3.0	21	3475	21	0	....	(†)	3.5	14	3275	14	0	....	(†)
4.0	138	4710	0	138	4710	....	4.0	101	3590	101	0	....	(†)
4.5	128	4360	0	128	4360	....	4.5	104	3780	104	0	....	(†)
5.0	391	4310	0	391	4310	....	4.75	182	4275	0	182	4275	....
5.25	20	4265	0	20	4265	....	5.0	443	3965	0	443	3965	....
5.5	538	4100	0	538	4100	....	5.25	57	4415	57	0	....	(‡)
5.75	93	4155	0	93	4155	....	5.5	211	3780	0	211	3780	....
6.0	779	4105	0	779	4105	....	6.0	58	3035	0	58	3035	....
6.25	7	3665	0	7	3665	....	6.25	4	3460	4	0	....	(*)
6.5	267	3620	0	267	3620	....	6.5	20	3370	18	2	2810	(‡)
6.75	195	3405	0	195	3405	....	6.75	3	3420	3	0	....	(*)
7.0	81	3715	24	57	3260	(‡)	7.0	4	2245	0	4	2245	....
7.5	29	2905	0	29	2905	....	11.0	1	1930	1	0	....	(*)
8.0	18	3370	18	0	....	(‡)							
8.5	2	3375	2	0	....	(*)							
Total	2707	....	65	2642	....	....	Total	1202	....	302	900	....	....
Both Periods: 1937 to March 1944													
4.5	278	4895	0	278	4895	....							
5.0	903	4470	128	775	4630	....	5.0	853	4220	0	853	4220	....
5.5	3943	4935	0	3943	4935	....	5.5	648	3875	0	648	3875	....
6.0	7373	4495	0	7373	4495	....	6.0	277	3607	0	277	3607	....
6.5	1136	3815	0	1136	3815	....	6.5	35	3325	18	17	3210	....
7.0	203	3365	24	179	3170	....	7.0	41	2635	0	41	2635	....
7.5	86	2985	0	86	2985	....							
Total	13922	....	152	13770	....	....	Total	1854	....	18	1836	....	....

*Insignificant representation.
†Inaccurate W/C data, water in aggregate ignored.
‡Special Cases—see text.

TABLE 6—RELATIONSHIP OF WATER-CEMENT RATIO AND CEMENT FACTOR TO COMPRESSIVE STRENGTH OF JOB CYLINDERS—28 DAYS—ROME AIR PORT

Cement Factor	Water—Cement Ratio						
	4.8	5.0	5.25	5.5	5.75	6.0	6.5
4.5	....	....	....	....	....	3475(88)	....
5.0	....	3960(26)	....	4190(21)	3980(111)	3345(10)	3870(22)
5.5	....	4210(59)	....	3560(78)	....	3045(57)	....
5.75	5040(141)	....	....	....	....	....	....
6.0	....	4570(51)	4690(55)	4275(59)	....	....	....

NOTE: Figure in parentheses following each compressive strength value indicates number of cylinders averaged.

By the exercise of the selectivity described, the values were obtained from which the relatively smooth curves shown in Fig. 2 (A) and (B) were constructed to indicate the relationship between compressive strength and water-cement ratio for the two periods. A comparison of the differences between the curves for comparable cement type for the different periods indicates the necessity of caution in combining the data to obtain the trend of the data as a whole. The combined weighted-average compressive-strength values tabulated in the lower part of Table 5 and plotted in Fig. 2 (C) and (D), represented only those water-cement ratios for which, for either type of cement, a representative number of specimens were tested during each period.

Of all of the available data in Table 5 the majority was used in preparing the graphs in Fig. 2 as the following figures indicate:

Graph	Percentage of all available data included	
Fig. 2 (A)	98	97
Fig. 2 (B)	91	
Fig. 2 (C) and (D)	93	

In view of the wide variety in type of structure built with cement containing Vinsol resin at the Rome Air Depot and the considerable amount of cement used therein, a summary of the strengths developed by the various mixtures used is given in Table 6. The next largest use of this type of cement was in the Stewart Field area, Newburgh, N. Y., where 419,913 barrels from 12 mills have entered six major projects to date. The details on strength developed by the concrete control cylinders in these projects are given in Table 7.

(c) *Type of cement.* In the foregoing reference has been made to apparent inaccuracies in reporting water-cement ratios. Similarly, it is believed that, in some cases, incomplete information was given on the type of cement used. Since the "Field Data" report form issued by this laboratory prior to the compilation of data did not require, specifically, that the presence of additions to the cement be recorded, it is believed that greater likelihood exists that specimens containing cement with

TABLE 7—MIXTURE DATA AND STRENGTHS OF CONTROL SPECIMENS AIR-FIELD PAVING—STEWART FIELD AREA

Project	W/C	C.F. bags	Compressive Strength of Test Cylinders at Ages Shown, psi. 7 day No.	7 day Max.	7 day Min.	7 day Aver.	28 day No.	28 day Max.	28 day Min.	28 day Aver.	90 day No.	90 day Max.	90 day Min.	90 day Aver.	Coarse Aggregate
NY-73	5.0	6.2	..	..	..	..	24	4250	2145	3490(*)	..	..	..	..	Gravel
	5.0	6.1	..	..	..	..	18	5605	2520	4130(†)	..	..	..	..	Gravel
NY-149	4.75	6.0	..	..	..	..	145	6640	2825	4205	..	..	..	..	Crushed
	5.0	6.0	..	..	..	..	53	5860	3125	4185	..	..	..	..	Dolomite
NY-544	5.0	6.1	82	4250	1645	3100	155	6250	2655	4600	11	6000	3535	4685	Dolomite
NY-193	5.0	6.0	..	..	..	..	25	4535	2590	3415	..	..	..	..	Dolomite
NY-324	4.8	6.7(‡)	..	..	..	..	38	5645	3820	4650	..	..	..	..	Dolomite
NY-325	4.8	6.5(‡)	..	..	..	..	50	5770	3395	4450	..	..	..	..	Dolomite
T & WA (‡)	4.9	6.2	82	4250	1645	3100	508	6640	2145	4305	11	6000	3535	4685	

Project	Flexural Strength (Modulus of Rupture) of Test Beams, psi. 7 day No.	7 day Max.	7 day Min.	7 day Aver.	28 day No.	28 day Max.	28 day Min.	28 day Aver.	60 day No.	60 day Max.	60 day Min.	60 day Aver.	90 day No.	90 day Max.	90 day Min.	90 day Aver.
NY-149	..	..	..	..	..	..	..	..	..	..	..	..	28	790	525	665
NY-544	77	904	350	560	131	904	416	690	10	820	630	745	32	895	525	720
T&WA (§)	77	904	350	560	131	904	416	690	10	820	630	745	60	895	525	695

*1942
‡High cement content made necessary by extreme harshness imparted by flat, angular and elongated stone.
§Totals and Weighted Averages
† 1943

Vinsol resin have been included in the plain cement group than vice versa. Since no specimens containing Vinsol resin were reported from the Baltimore District, a spot check was made of data from that source to endeavor to uncover an example of this condition. In the case of one project (BA-31), the cement shipment records indicate the exclusive delivery of cement containing Vinsol resin to the project in cars, the numbers of which are given in the "Field Data" reports as sources for cement used in the cylinders. The "Field Data" reports however, failed to state the fact that the cement contained Vinsol resin.

Airport evaluation program

A total of 236 slabs of concrete, varying from 3 ft. by 3 ft. to 6 ft. by 8 ft. were removed from the runways or aprons at 26 airports in the northeastern portion of the United States, in connection with an airport pavement evaluation program. The slabs, removed after periods in place varying from six weeks to several years, afforded opportunity to obtain additional information on the strength of pavement concrete made with cement containing Vinsol resin. Also, some indication of the effect of Vinsol resin in the cement as compared to plain cement and the effect of the type of coarse aggregate was developed. These relationships however, must be interpreted with caution, because the various conditions of cement type and coarse aggregate type seldom existed on the same project. Thus the important effects of brand of cement, water-cement ratio, cement content, fine aggregate, weather conditions, age, type of curing, contractor's equipment and personnel, and engineering control were not and could not be evaluated. For instance, the abnormal air-entrainment developed on two of the jobs (Grenier Field and Trumbull Airport) in which gravel was used probably would not have been different had crushed stone been used. In another case, extremely variable water control was indicated on one of the older plain cement jobs where gravel was used as coarse aggregate. It just happens that no such major variables are known to have occurred on the projects in which crushed stone was used as the coarse aggregate.

The concrete slabs were trucked to this laboratory where 6 in. by 36 in. beams were extracted by sawing and 6 in. diameter cores were extracted by means of a diamond core drill. The depths of the beams and lengths of the cores, being functions of the depth of the slabs, varied from 4 in. to 11 in. Where the slab thickness exceeded 6 in., the excess was removed by sawing. Where the thickness was less than 6 in. no adjustment was made other than a correction for h/d in the cores. Cores were tested at the greatest lengths practicable after removing major irregularities by sawing. Some of the slabs contained wire-mesh reinforcement, but the effects of such reinforcement on the strengths of the beams, cubes and cores were minimized if not removed entirely by selective sawing and drilling. In

general, 5 beams and 3 cores were extracted from each slab. After the beams had been tested in flexure, compression tests were made on the broken pieces (modified cube). The tests on the beams, modified cubes and cores were conducted in accordance with prevailing A.S.T.M. Standards with the following exceptions:

(a) The total breaking loads on the modified cubes were corrected by the factors given in A.S.T.M. C 42-42 (Sec. 7) to eliminate the effect of varying depths of specimens on the apparent compressive strength.

(b) The beams tested in flexure in the later portion of the program were tested with the surface as cast in compression. This was done on orders from the Office, Chief of Engineers. There was no detectable evidence that the orientation of the beam in the testing machine affected the apparent flexural strength of the concrete.

Character of concrete. It was not possible to obtain a precise description of the proportions used in all of the concrete pavements, but it can be stated that the basic policy of the specifications for this class of work was adhered to in a majority of cases. On this basis, the concrete in the pavements contained coarse aggregate of 2 in. maximum size; an average cement content of 6.0 bags per cu. yd., varying in certain cases, from 5.5 to 6.7 bags per cubic yard (with the higher factors being used, generally, in connection with crushed stone coarse aggregate); a slump averaging 2½ in., and water-cement ratios varying from a minimum of 4.7 gals. per bag for cement containing Vinsol resin to a maximum of 6.3 gals. for plain cement. The age of the concrete slabs varied widely—from a minimum of 45 days to a maximum of several years. However, the influence of the age factor is not considered to be significant because all types of concrete are represented in essential equality in the age range.

Flexural strength—sawed beams. The results obtained from the 1524 flexural tests (third point loading) on sawed beams are summarized in Table 8 by projects, basic types of cement and basic types of coarse aggregate. These same data are abstracted in Table 9.

Compressive strength—modified cubes. The results obtained from the 1441 compressive tests made on modified cubes from sawed beams are summarized in Table 8 by projects, basic types of cement, and basic types of coarse aggregate. These data are abstracted in Table 10.

Compressive strength—cores. The results obtained from the 437 cores drilled (diamond bit) from the slabs are summarized in Table 8 by projects, basic types of cement and basic types of coarse aggregate. These data are abstracted in Table 11.

Miscellaneous pavement cores. In addition to the cores taken from the Pavement Evaluation slabs, a total of 2177 6-inch diameter cores were extracted from pavements in situ at military airports in this region. These cores were tested in compression in a moist condition after their bulk displacement had been determined. The strength data and certain

TABLE 8—SUMMARY OF STRENGTH AND DENSITY DATA OBTAINED FROM CONCRETE SLABS TAKEN FROM AIRFIELD PAVEMENTS

Field	No. of Slabs Tested	Flexural Strength, psi.				Compressive Strength, psi.								Density—Cores			Ref. See Foot Notes
		Sawed Beams				Modified Cubes from Sawed Beams				6-in Diamond Drilled Cores				Actual Unit Weight lb./cu. ft.			
		No. of Tests	Max.	Min.	Aver.	No. of Tests	Corrected to h/w = 2 Max.	Min.	Aver.	No. of Tests	Corrected to h/d = 2 Max.	Min.	Aver.	Max.	Min.	Aver.	
Plain Portland Cement—Gravel Corase Aggregate																	
Millville	6	54	810	460	635	54	7500+	4145	5655	18	7555	5045	5970	153.1	147.5	151.2	
Dover	8	77	760	450	625	76	6825	4670	5795	24	6395	4840	5550	153.7	149.4	151.2	(1)
New Castle	2	18	735	530	645	18	5425	4425	4945	6	5320	4565	5100	——	——	——	
Dow	6	59	1105	610	800	59	7930	4340	5525	18	6200	4215	5400	153.7	146.2	150.3	
Presque Isle	6	50	825	610	700	50	6175	4370	5285	18	6530	4515	5475	151.2	147.5	148.3	
Mitchell	28	272	1035	220	645	260	7960	2070	5450	83	8350	2245	5680	159.4	136.7	147.3	(2) (3)
Green	11	89	830	460	670	75	6870	3015	5445	26	9215	3200	6675	154.4	145.6	151.0	
Westover	2	14	760	515	645	10	6200	4415	5105	0	—	—	—	——	——	——	
Stewart	1	10	1150	735	900	10	5630	3520	4870	3	6090	5950	5995	153.7	146.9	151.4	(4)
Bedford	4	40	965	590	780	20	5745	3330	4820	12	8345	6025	6940	144.1	143.1	143.6	(5)
Grenier	3	30	810	455	635	30	6285	3495	4925	9	6680	5180	5915	165.6	150.0	152.5	(1)
Totals and Averages	77	713	1150	220	700	662	7960	2070	5345	217	9215	2245	5825	159.4	136.7	149.7	
Revised (See Note 8)	74	683	1150	220	705	632	7960	2070	5290	208	9215	2245	5865	159.4	136.7	149.4	(8)
Plain Portland Cement—Crushed Stone Coarse Aggregate																	
Middletown	4	40	1095	770	980	30	6525	4670	5640	10	7985	4025	5815	156.2	151.2	154.0	
Suffolk	10	92	995	535	870	92	7155	3910	5460	29	6595	4450	5660	155.0	148.1	151.3	
Westover	3	26	900	620	750	25	7175	4090	5640	0	—	—	—	——	——	——	
Totals and Averages	17	158	1095	535	865	147	7175	3910	5580	39	7985	4025	5735	156.2	148.1	152.7	
Cement with Vinsol Resin—Gravel Coarse Aggregate																	
Rye Lake	11	101	750	480	605	103	5895	3550	4795	33	6360	3875	5200	151.9	141.9	148.2	
Fort Dix	7	70	780	470	600	70	5925	2460	4630	21	6105	3800	5010	147.5	144.4	146.1	
Trumbull	6	60	710	275	530	60	6375	1775	4255	0	—	—	—	——	——	——	
Stewart	5	50	1125	540	730	40	6675	2885	4770	15	6290	3745	5150	146.4	135.4	141.9	(2) (6)
Grenier	5	48	680	220	470	48	5925	1075	3270	15	6680	1520	3550	151.2	128.9	140.9	
Totals and Averages	34	329	1125	220	585	321	6675	1075	4345	84	6680	1520	4785	151.9	128.9	144.3	
Revised (8)	29	281	1125	275	615	273	6675	1775	4615	69	6360	3745	5120	151.9	135.4	145.4	(8)

—Table 8 continued on opposite page

TABLE 8—Continued from opposite page

Field	No. of Slabs Tested	Flexural Strength, psi.				Compressive Strength, psi.								Density—Cores			Ref. See Foot Notes
		Sawed Beams				Modified Cubes from Sawed Beams				6-in Diamond Drilled Cores				Actual Unit Weight			
		No. of Tests	Max.	Min.	Aver.	No. of Tests	Corrected to h/w = 2			No. of Tests	Corrected to h/d = 2			lb./cu. ft.			
							Max.	Min.	Aver.		Max.	Min.	Aver.	Max.	Min.	Aver.	
Cement with Vinsol Resin—Crushed Stone Coarse Aggregate																	
Stewart	10	99	820	650	710	89	5845	3880	4820	30	6450	4070	5115	156.2	149.4	152.1	
Galeville	7	70	940	660	785	70	7165	5250	6035	21	8485	5545	6565	154.4	146.2	150.5	
Montgomery	7	67	995	520	795	67	7260	3780	5840	21	7890	4750	6675	156.2	148.7	154.0	
New Hackensack	4	40	1010	710	850	40	5925	4200	5045	12	6970	5720	6335	153.7	149.4	151.4	
Westover	4	17	1030	580	865	14	7080	3935	5740	0	—	—	—	——	——	——	
Rome	3	31	960	730	845	31	6845	4595	5915	3	6705	4385	6100	151.9	150.0	150.6	(7)
Totals and Averages	35	324	1030	520	810	311	7260	3780	5565	97	8485	4070	6160	156.2	146.2	151.7	

NOTES:
(1) Includes two locations in which the pavement contained wire-mesh reinforcement.
(2) Includes one location in which no modified cube tests were made because beams were less than 5-in. thick.
(3) Includes 11 locations in which the pavement contained wire-mesh reinforcement.
(4) High-Early-Strength cement used.
(5) Includes two locations in which no modified cube tests were made because beams were less than 5-in. thick.
(6) Includes one location in which 1 per cent $CaCl_2$ was added in addition to Vinsol resin.
(7) Includes three locations in which the pavement contained wire-mesh reinforcement.
(8) Omitting data from Grenier Field.

TABLE 9—MODULUS OF RUPTURE—SAWED BEAMS

Third point loading; Age: Greater than 45 days

	Plain Cement		Cement + V.R.	
	Gravel	Stone	Gravel	Stone
No. of tests	713	158	329	324
Max. strength, psi	1150	1095	1125	1030
Min. strength, psi	220	535	220	520
Aver. strength, psi	700	865	585	810
Mean aver. (gravel & stone) psi	780		700	
Mean aver. strength of concrete containing V.R. as *per cent* of concrete containing plain cement	90			
Weighted aver. (gravel & stone)	730		695	
Weighted aver. strength of concrete containing V.R. as *per cent* of concrete containing plain cement	95			

TABLE 10—COMPRESSIVE STRENGTH—MODIFIED CUBES

Age: Greater than 45 days

	Plain Cement		Cement + V.R.	
	Gravel	Stone	Gravel	Stone
No. of tests	662	147	321	311
Max. strength, psi	7960	7175	6675	7260
Min. strength, psi	2070	3910	1775	3780
Aver. strength, psi	5345	5580	4345	5565
Mean aver. (gravel & stone) psi	5460		4955	
Mean aver. strength of concrete containing V.R. as *per cent* of concrete containing plain cement	91			
Weighted aver. (gravel & stone) psi	5390		4945	
Weighted aver. strength of concrete containing V.R. as *per cent* of concrete containing plain cement	92			

statistical constants are abstracted in Table 12. It will be observed that the general average strength of the cores containing Vinsol resin is 93.5 per cent of the cores containing plain cement in spite of inclusion in the average of cores taken from Grenier and Trumbull where the air content was abnormally high. It will be observed, further, that the standard deviation and coefficient of variation of the strengths of the cores containing Vinsol resin are somewhat lower than they are for the strengths of cores containing plain cement.

The frequency distribution of the compressive strengths of the cores from four projects is plotted in Fig. 3. These data will be referred to

TABLE 11—COMPRESSIVE STRENGTH—AIRPORT EVALUATION CORES

Age: Greater than 45 days

	Plain Cement		Cement + V.R.	
	Gravel	Stone	Gravel	Stone
No. of tests	217	39	84	97
Max. strength, psi	9215	7985	6680	8485
Min. strength, psi	2245	4025	1520	4070
Aver. strength, psi	5825	5735	4785	6160
Mean aver. (gravel & stone) psi	5780		5470	
Mean aver. strength of concrete containing Vinsol resin as *per cent* of concrete containing plain cement	95			
Weighted aver. (gravel & stone) psi	5810		5520	
Weighted aver. strength of concrete containing Vinsol resin as *per cent* concrete containing plain cement	95			

later in connection with a discussion of the strength-density relationship.

Discussion. The data on strength presented above should be evaluated with the following factors in mind:

A very large number of tests (22,398) is included; all made by the same laboratory.

The field control specimens were molded from concrete actually placed in structures and stored initially in the field under varying conditions of temperature and moisture.

Other specimens were removed from the finished structures and, therefore, subject to the variables of composition and curing which are not characteristic of laboratory test specimens.

Data on water-cement ratio and type of cement were obtained from field reports.

With the foregoing in mind, the test data indicate the following:

(1) An approximate correlation exists between the compressive strength and reported water-cement ratio of job-made cylinders. This relationship of strength to water-cement ratio is in reasonable accord with that assumed by the laboratory as representing the strengths which might normally be expected.

(2) A lower order of compressive strengths was indicated for concrete placed in 1943-44 than for that placed prior to 1943.

This is regarded as reflecting (1) the use of more coarsely ground cement manufactured under WPB Limitations Order L-179, and (2) the effect of emergency conditions on the quality of field control; especially, on the accuracy with which specified water-cement ratios were maintained and reported in practice, since the decrease in compressive strengths was

TABLE 12—SUMMARY OF COMPRESSIVE STRENGTH DATA—MISCELLANEOUS DRILLED CORES

Concrete Pavements

Project Name	No. of Cores	Compressive Strength, psi. Max.	Min.	Aver.	Stan. Dev. psi.	Coeff. of Var. —
CEMENT WITH VINSOL RESIN						
Westover Field	20	8755	4950	6765		
Trumbull Airport	22	5000	1560	3210*		
Grenier Field	100	6490	1415	3570*		
Fort Dix Airfield	44	6520	2430	4015		
Rome Air Depot	232	8480	4040	6575	699	10.6
Rye Lake Airport	165	6025	2575	4260		
Stewart Field (Apron)	86	6445	2255	4390		
Stewart Field (Runways)	565	8515	3105	5340	972	18.2
Galeville Airfield	310	7985	3785	6130	929	15.2
Montgomery Airfield	203	6540	3480	4820	586	12.2
New Hackensack Airfield	40	6065	3885	4900		
West Point Roads	25	7190	1940	4655		
Syracuse Air Base	21	7535	4970	6365		
Totals and Averages	1924	8755	1415	5290	861	15.4
PLAIN CEMENT						
Westover Field	15	8615	3200	6480	1854	28.6
Grenier Field	12	6275	3230	5115	908	17.7
Bedford Airport	10	7230	3510	5765	1230	21.3
Suffolk Airport	20	5360	3800	4580	479	10.5
Dow Field	30	7065	4880	5980	573	9.6
Mitchel Field	152	7835	2315	5660	858	14.4
Rome Air Depot	14	8120	4915	6560	1481	22.3
Totals and Averages	253	8615	2315	5655	1055	17.8

(*) Abnormally high air content.

greater in the lower range of reported water-cement ratios than in the normal or higher range.

The anomalies in the data indicate the importance of proper direction of inspecting personnel to determine and report the *actual net water-cement ratio* entering the concrete sampled for testing.

(3) Since the largest number of specimens containing Vinsol resin were at a water-cement ratio one gallon per bag less than that at which the largest number of plain cement specimens are found, and since the cement factors and slumps are similar, it is concluded that the field evidence substantiates the laboratory experience that, when cement con-

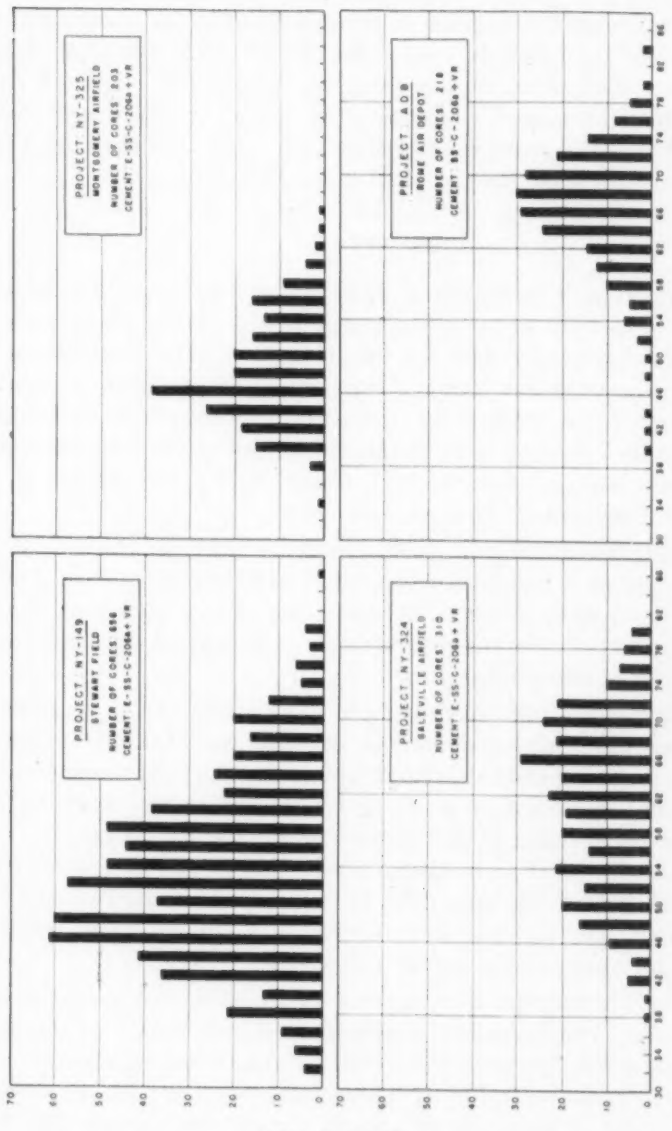

Fig. 3—Frequency distribution of compressive strengths of concrete cores drilled from airfield pavements.

taining Vinsol resin is used with a water-cement ratio one gallon per bag lower than would be required for plain cement, an equivalent degree of workability can be obtained with similar cement factors.

(4) When recognition is given to the statement in (3) above, and the strength values for concrete containing Vinsol resin are plotted to a water-cement ratio scale increased by one gallon per bag from those reported to have been used, and when abnormal air contents are avoided, it is evident that the strengths developed by concrete containing Vinsol resin are comparable to those of plain cement concrete.

DENSITY

No regular attempt having been made by the field inspection forces to determine the amount of air entrainment in the plastic concrete, fragmentary data only are available on the air content of the concrete on the numerous projects where cement containing Vinsol resin was used. Available data came, principally, from the projects where spot checks were made at the specific request of this office. Except for the two projects mentioned available evidence indicates that the air content was in the range 3.5—7.0 per cent of the concrete volume.

Recent instructions require determination of the air content of the plastic concrete as a regular and routine procedure. However, for the purpose of this paper, principal dependence must be placed on density determinations on hardened specimens for information on the amount and uniformity of air-entrainment.

The density of the hardened concrete was determined in conjunction with compressive strength tests on 6-in. diameter cores taken from airport pavements. Determinations were by displacement and values expressed in terms of weight in pounds per cubic foot. These data are summarized in Table 8, for cores taken in the pavement evaluation program, and in Table 13 for additional cores taken in the field from pavements in the Stewart Field, New York, area. The air contents shown in Table 13 were calculated from the theoretical unit weight of the concrete, based on mixture proportions, and the actual unit weight.

Tables 8 and 13 indicate the degree of variation in unit weight in each of the projects. The frequency distribution of unit weight of the cores, taken from the four projects from which the greatest number of cores were extracted, is shown in Fig. 4. These data, together with the strength data shown in Fig. 3, were used to prepare the scatter diagrams (Fig. 5) as the relationship between compressive strength and actual unit weight of the concrete tested. The correlation coefficients calculated from these data are indicated in the illustration by numerical values and by regression lines drawn through the scatter diagrams. Additional strength-density

TABLE 13—DENSITY DATA—DRILLED CORES FROM AIRFIELD PAVEMENTS IN STEWART FIELD AREA

Cement: E-SS-C-206a + VR

Project	W/C	C.F.	Actual Unit Weight, lb./cu.ft.				Approx. Th. Unit Wt., lb./cu.ft.	Calculated Average Air Content per cent
			No.	Max.	Min.	Aver.		
Stewart Apron (NY-73)	5.0	6.1	75	148.1	132.5	142.2	149.8	5.1
Stewart Runways (NY-149)	4.8	6.0	529	158.1	146.9	152.6	159.3	4.2
New Hackensack (NY-193)	5.0	6.0	65	157.9	146.0	151.6	158.8	4.5
Galeville Runways (NY-324)	4.8	6.7	243	156.2	143.7	150.5	154.9	2.8
Montgomery Runways (NY-325)	4.8	6.5	217	157.6	144.4	151.6	157.3	3.6
Area Totals and Average	...	...	1129	.....	.....	.....	.....	4.0

*Some specimens tested in compression without unit weight determination; some cores upon which unit weight determinations were made were tested other than in compression.

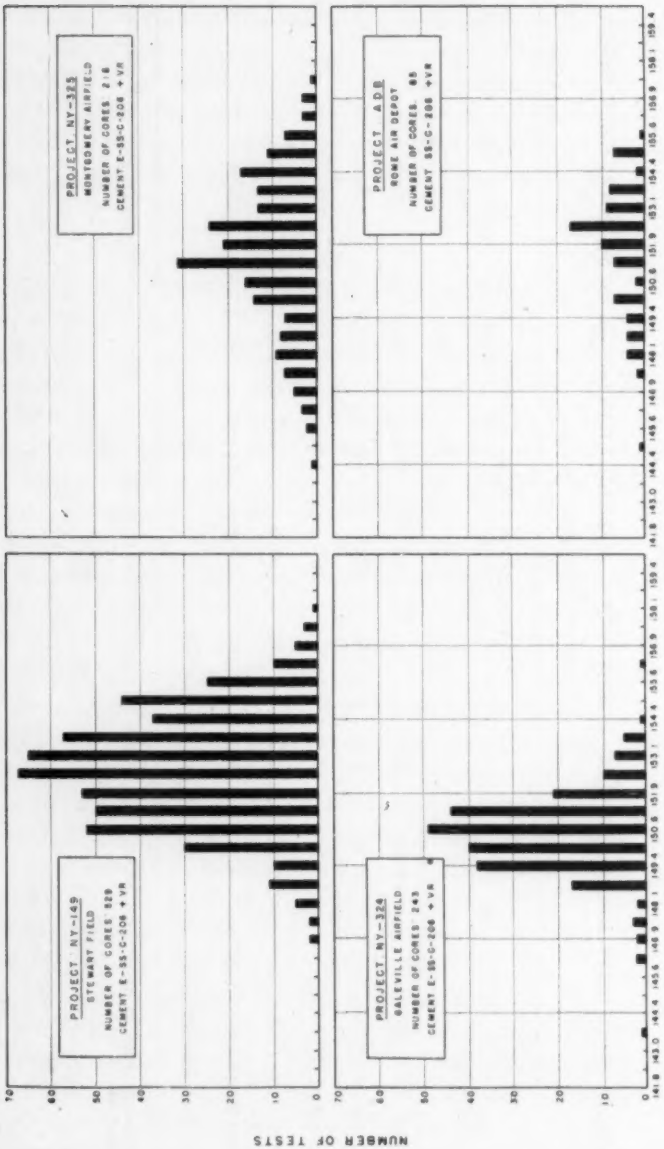

Fig. 4—Frequency distribution of unit weights of concrete cores drilled from airfield pavements.

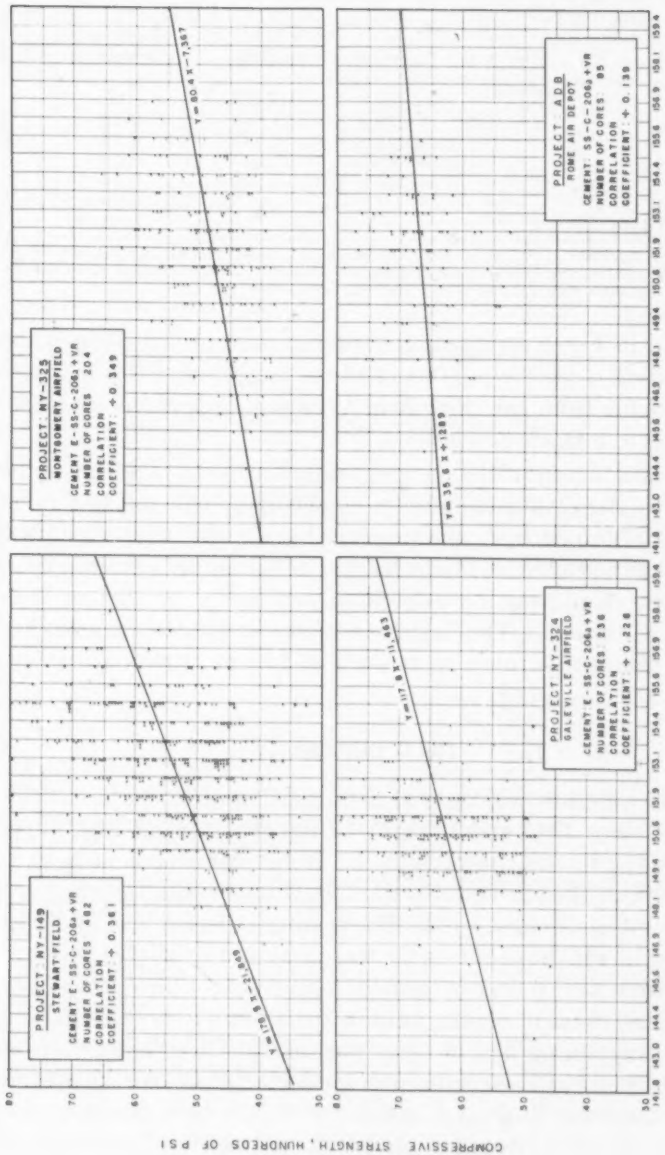

Fig. 5—Relationship between compressive strength and actual unit weight, concrete cores.

data are plotted in Fig. 6 and 7 based on cores extracted from Grenier Field and Trumbull Airport, respectively.

The regression equations and coefficients of correlation are:

Source	Regression equation	Coeff. of correlation	Number
B. of R. (*)	$Y = 197\ X - 24{,}400$	0.60	46
NY-149	$Y = 178.5\ X - 21{,}849$	0.36	482
NY-324	$Y = 117.8\ X - 11{,}463$	0.23	236
NY-325	$Y = 80.4\ X - 7{,}367$	0.35	204
ADB	$Y = 35.6\ X - 1{,}289$	0.14	85
Grenier Field	—	0.90	97
Trumbull Airport	—	0.94	28

(*) Fig. 3, p. 1005 Proc., A.S.T.M. v. 43 (1943) Wing, S.P.; Jones, Valens; and Kennedy, R.E. "Simplified Test for Evaluating the Effectiveness of Concrete Mixers". Data represents plain cement and are included for comparison with the mixtures containing Vinsol resin.

The data in Fig. 8 represent the relationship, or lack thereof, between the air content of plastic mixtures and the compressive strength of control cylinders made from the same field batches. This is the single instance in which such data are available.

These data confirm the general relationship which exists between compressive strength and density of concrete, but the low correlation coefficients obtained from the projects having the most significant data indicate that variable or low strengths obtained in concrete containing Vinsol resin should not be attributed arbitrarily to high or materially varying air content. The influence of water control, segregation, and degree of compaction play an important role in attaining high and uniform strength.

FIELD DURABILITY

The only tangible evidences of deterioration of concrete in field structures built since 1940 and brought to the attention of this office are in surface scaling in the runways at the Rome Air Depot and, to a much lesser extent, in the apron at Stewart Field, and rather severe cracking of the pavement at Grenier Field. The scaling at Rome was limited mainly to the area in which plain cement was used and in a large portion of the area in which one brand of cement containing Vinsol resin was used. Small isolated areas of scaling exist in the areas where three other brands of cement containing Vinsol resin was used. The scaling at Rome was made the subject of a special report entitled "Rome Air Depot—Surface Scaling of Concrete Runways" which was circularized within the Engineer Department by this laboratory under the date of October 1943.

The conclusion drawn in the report was that the scaling of the pavement is primarily of the "manipulation" type and the principal influences in its development, in their order of importance, were:

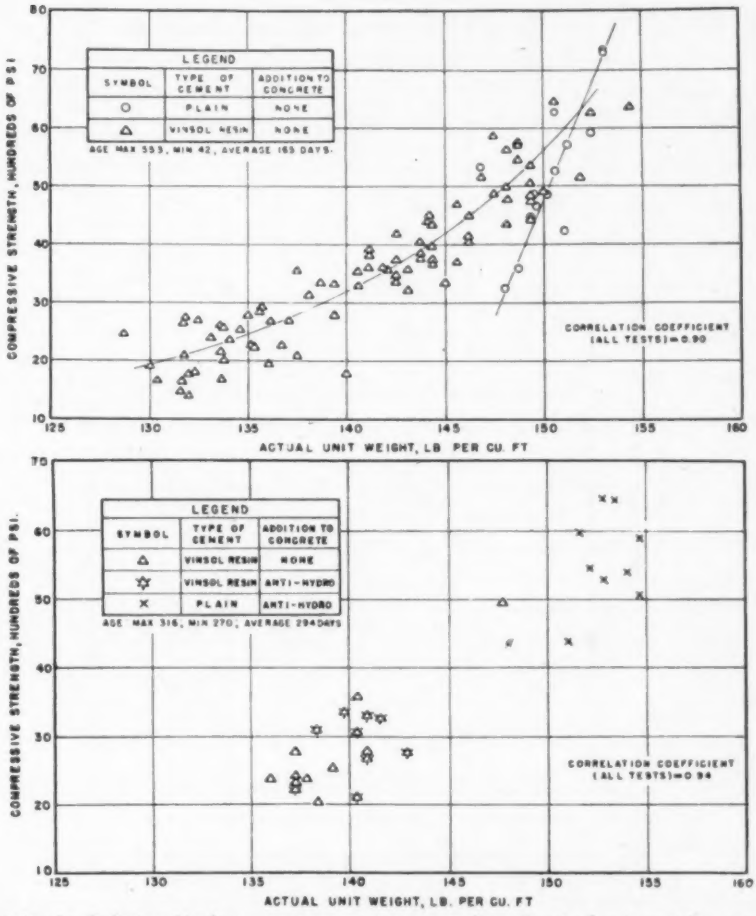

Fig. 6 (top)—Relationship between compressive strength and actual unit weight, concrete cores, Grenier Field, Manchester, N. H.

Fig. 7 (bottom)—Relationship between compressive strength and actual unit weight, concrete cores, Trumbull Airport, Groton, Conn.

(1) Excessive manipulation of the highly plastic mixture during the finishing operation.

(2) Exposure to frequent and severe freezing and salt applications very shortly after the end of the 14-day curing period.

(3) Inadequate air-entraining reaction of the Vinsol resin in one cement, or basic inferiority in that cement.

The scaling has been progressive in extent. The condition of the scaling in July 1943 and in October 1944 is summarized in Table 14. These data indicate clearly that despite the considerable extent of scaling in the con-

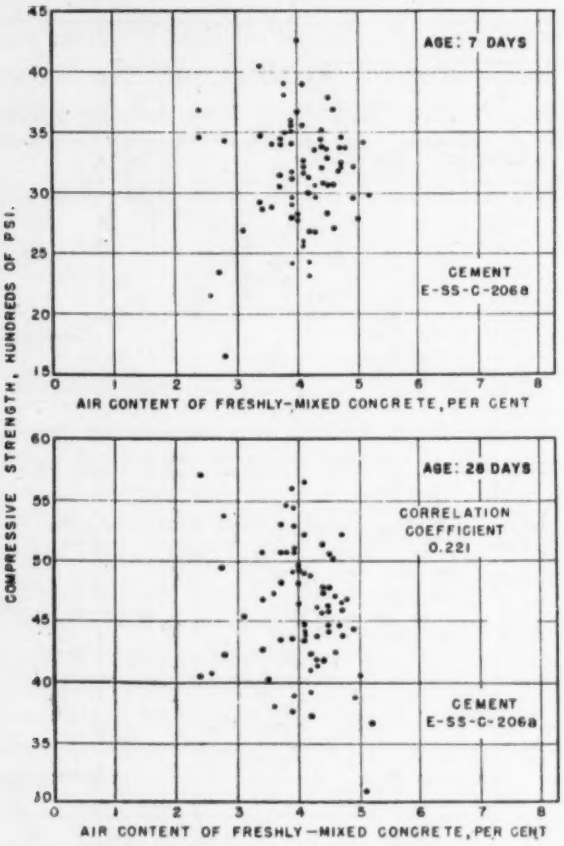

Fig. 8—Relationship between compressive strength of that cylinders and air content of freshly mixed concrete—N Y 544. Additional paving—Stewart Field landing mat.

crete containing Vinsol resin, this type of concrete is greatly superior to concrete made with the plain cement.

The scaling which has occurred in a small portion of the section of Stewart apron placed in 1942 varies from insignificant to moderate and appears to have occurred only in areas placed during or just prior to freezing temperatures without subsequent protection, or in areas where the concrete was placed on very wet and soft sub-grades or during rain.

The cracking at Grenier Field is believed to be due (1) to structural weakness in the concrete due to an excessive air content (2) to the concrete having been placed on a frozen sub-grade, and (3) to excessively heavy equipment running on surfaces placed on deep fill.

The subject of the durability of concrete made with and without Vinsol resin will be discussed in a separate paper.

TABLE 14—ROME AIR DEPOT

Summary of Pavement-Surface Scaling

Cement Brand	Addition	Scaling to July 1943 Blocks* Scaled—%		Scaling to Oct. 1944 Blocks* Scaled—%		Increase in Scaling Blocks Scaled—%	
		Total (A)	Total (B)	Total (A)	Total (B)	Total (A)	Total (B)
1	VR†	50.8	0.73	66.5	0.83	15.7	0.10
2	VR	1.6	0	6.0	0.10	4.4	0.10
3	VR	1.6	0	15.4	0.50	13.8	0.50
4	VR	1.4	.0.1	5.6	0.40	3.2	0.30
1	—	93.5	19.4	96.8	37.10	3.3	17.70
1	Nat‡	83.3	8.1	99.1	18.3	15.8	10.20
2	VR+Nat**	0	0	5.4	0	5.4	0

*Block = 12.5 ft. by 20 ft.
†VR = Flake Vinsol resin interground at mill.
‡Nat. = 1 bag Natural cement replacing 1 bag of plain portland cement in mix.
**VR + Nat. = 1 bag Natural cement replacing 1 bag portland cement containing VR in mix.
(A) = Thin "manipulation" type scale.
(B) = Deeper progressive scale.

TIME AND TYPE OF MIXING

Attention was focused on the possible effect of time and type of mixing on the air content and strength of concrete containing Vinsol resin when it was determined that transit mixing (not agitation) of 20 minutes duration was used at Grenier Field during the period in which excessively high air contents were obtained. Information solicited from other agencies at that time indicated that mixing in transit mixers with sealed drums might be the cause of excessive air-entrainment in the concrete. A survey of military projects, using transit mixers and cement containing Vinsol resin, in this region, produced vague and conflicting information due to the incompleteness of records and the use of many different types of transit-mixers, each type differing somewhat in drum design, discharge mechanism, condition of blades, venting or water control. Additional variables were time of agitation (as distinguished from time of mixing), time of mixing and length of haul, plus brand of cement and type of aggregate.

An investigation of the conditions of mixing at Grenier Field in the spring of 1943 included the use of several brands of cement in which the Vinsol resin * content varied. The sand content of the mixture was varied

*The flake or un-neutralized form.

from 35 to 25 per cent by weight of the aggregate. The mixing was done in the contractor's horizontal-discharge, closed-drum type of transit mixer. The time of mixing and load in the mixer were varied. In this investigation, it was observed that;

(a) Time of mixing had no great effect on the amount of entrained air, the air content of all mixtures being abnormally high irrespective of mixing time.

(b) Load in the mixer, up to the limit specified by the manufacturer, had no great effect on the amount of entrained air.

(c) The brand of cement had more effect upon the amount of entrained air than any other factor when Vinsol resin was present, .

(d) Reducing the amount of Vinsol resin to the minimum permitted by the specifications* was inadequate to hold the air content in the concrete to the desired limit with the cements and mixers used.

(e) The air content was affected appreciably by the sand content of the mixture (increasing in proportion with the sand content).

(f) The homogeneity of the mixture obtained with the transit mixers used was unsatisfactory without excessive mixing time (10-15 minutes) or unless the drum load was considerably reduced from the rated capacity.

Additional but less formal investigations made subsequently at various other projects using transit mixers indicated that the air content might increase with mixing time to a varying extent. No data were obtained on the effect of mixing time on air content of concrete mixed in pavers or in stationary mixers because the mixing time was constant at 1 or 1½ minutes with this type of equipment. However, laboratory studies will be discussed in a subsequent paper.

The large amount of paving since 1941 under emergency conditions resulted in the use of many types and conditions of paving mixers and transit mixers; with pavers and transit mixers occasionally operating together and in at least one case supplying concrete to parallel strips of pavement. Full advantage could not be taken of the opportunity for careful recording of results due to shortage and inexperience of inspectors and the general rush of construction, but the overall experience with the various types of mixers handling concrete with plain cement or with cement containing Vinsol resin may be summed up, as follows:

(a) Less difficulty was experienced with pavers than with any type of transit mixers in obtaining the quality of concrete desired — with or without Vinsol resin.

(b) Due, possibly, to fortuitous circumstances; no concrete placed with pavers developed excessive air content or strengths lower than specified. The mixing time for pavers was 1½ minutes.

*Equivalent to A.S.T.M. C 175-42 T.

(c) Due, possibly, to inadequate maintenance conditions, occasional difficulties in consistency control were experienced with dual-drum pavers when portions of the second batch intermingled with the first.

(d) Good results were obtained throughout the placement period in some projects where transit mixers were used, especially where the mixing was accomplished after arrival of the truck at the point of deposit and vent holes were kept open and mixing time was limited. At one such job, Fort Dix, the taxiways were of concrete mixed in pavers and the aprons and hardstandings constructed, simultaneously, of concrete from transit mixers. Efforts toward control of air content and other characteristics of the mixture were particularly vigorous on this job, and the mixing time in transit mixers was held to a minimum. The average data for the two projects, based on information from field reports, indicate the results obtained were similar (Table 15).

(e) The most frequent complaints on the performance of transit mixers concerned inability to control water content with resultant variations in consistency and strength.

(f) Performance in transit mixers appears to be improved when the slump is increased to about 4 inches, permitting the mixture to surge in the drum; and when the load is reduced from 57.5 per cent to approximately 40 per cent of the volume of the drum.

(g) It has not been practicable to establish that venting of the drum has any effect on the amount of air entrained in the concrete.

(h) In general, the high-discharge type of transit mixers appears to handle the drier consistency mixtures more effectively than the horizontal-discharge type.

(i) Study of the mixing action in various types of transit mixers indicates that, as a class, they were not designed to produce rapid mixing of low-consistency concrete. The concrete occupies space greater than the radius of the drum when the load is up to the manufacturer's normal recommendation of 57.5 per cent of the volume of the drum. The spiral mixing blades are less than ½ radius deep, therefore, the slow revolution of the drum results in general folding action with a slight forward component when the relatively dry consistency typical of paving mixtures is used. This is in contrast to the paving mixer in which the mixing action is by "bucket throw" and the mixing blade area is considerably greater, the speed of revolution is faster and the volume ratio of concrete to drum is much less than in transit mixers. The rolling and folding action in a transit mixer appears to entrap rapidly a considerable amount of air even though the uniform distribution of the mixture components may not be complete. The writer concludes that the general tendency of transit-mixed

TABLE 15—FORT DIX PAVEMENT DATA

	PH-17—Taxiways	PH-12—Aprons and Hardstandings
Mixers:	Pavers (27E and 34E) Dual-drum	Transit Mixers—(Rex, Jaeger, Smith) horizontal discharge. Quadruple vents kept open and wash water wasted.
Mixing time:	1½ minutes	Aver. 5 min. (3-10 min.)
Design Mix:	1:1.84:4.07	1:1.84:3.97
Slump:	2—2½ in.	2—3 in.
Cement:	SS-C-206a+VR (3 brands)	SS-C-206a+VR (1 brand)
Fine aggregate:	Natural sand (Aver. F.M.: 2.26)	Natural sand (Aver. F.M.: 2.26)
Coarse aggregate:	Uncrushed gravel: 2 in. max.	Uncrushed gravel: 2 in. max.
Design W/C:	4.8 gal. per bag	4.8 gal. per bag
Aver. cement content:	5.8 bags/cu. yd.	5.9 bags/cu. yd.
Air content, per cent:	Aver. 3.7 (3-4.5)	Aver. 4.4 (2.5-5)
Strengths developed:		
Field: (a) Compressive: (drilled cores):	4840 psi. (13)*	4425 psi. (6)
(mod. cubes):	4995 psi. (40)	4295 psi. (20)
(b) Flexural: (sawed beams):	635 psi. (40)	595 psi. (20)
Control: (a) Compressive: (cylinders—28 days):	3355 psi. (101)	3800 psi. (52)
Standard deviation:	560 psi.	660 psi.
Coeff. of variation	16.7%	17.4%
(b) Flexural: (beams—6 in. x 6 in.):	490 psi. (190)	505 psi. (83)
Standard deviation:	67 psi.	93 psi.
Coeff. of variation	13.7%	18.6%
Aver. coeff. of Variation All specimens:	15.2%	18.0%

*Figure in parentheses indicate number of tests.

paving mixtures containing Vinsol resin to have abnormally high air contents is a function of *type* of mixing action rather than *time* of mixing although prolonged mixing may serve further to increase the air content.

PLACING AND FINISHING

Comparative information on placement and finishing of concrete containing Vinsol resin, difficult at best to present in a quantitative manner, is in the writer's experience made more difficult by the fact that the occasions were infrequent when concrete containing plain cement and cement containing Vinsol resin were placed simultaneously or under precisely similar conditions. However, it is the consensus of the project engineers, contractors and inspectors with whom the subject has been discussed that concrete containing Vinsol resin compares with concrete containing plain cement about, as follows:

(a) The slump of the mixture is more sensitive to change in water content.

(b) A mixture having a consistency of 2-in. slump is more readily placeable, that is, more mobile.

(c) Segregation of any component (water, sand or coarse aggregate) is less likely to occur, and when it occurs it is less serious in extent except that segregation of fine aggregate is most likely to occur, because it is most likely to be present in excess of the optimum quantity. In this connection, it should be recalled that the spheroids of entrained air act as a fine aggregate, therefore, the amount of sand usually considered normal for plain cement concrete will be excessive in concrete containing Vinsol resin by about the amount of the entrained air.

(d) Consolidation by vibration is effective, but any type of consolidation should be accomplished only to the degree necessary to effect compaction and with more care than is usually devoted to this important operation.

(e) The concrete mixture is "sticky" and there is a tendency for the surface to tear under the influence of a mechanical screed or finishing machine especially when the slump of the mixture is less than 2 in. There is the further common complaint from contractors using Vinsol resin for the first time that final finishing is more difficult to perform. Actually, the difficulties referred to are due to the basic difference in the nature of the material and can and have been effectively overcome by experience. The tendency to tear can be overcome by shortening the forward motion of the mechanical screed or finisher for each transverse stroke or by maintaining a slump of 2½ to 3 inches, or both. The difficulties in final finishing can be overcome by performing the work earlier than is done normally with plain cement, that is; by materially closing the gap between the preliminary mechanical and final finishing operations. This is made practicable and necessary by the reduced bleeding of Vinsol resin concrete. It appears to be the consensus of those who have learned to handle concrete containing Vinsol resin that the difficulties of finishing

are no greater than with plain cement and that the shorter lag between screeding and final finishing results, usually, in an over-all economy.

Of *paramount* importance in the finishing of V. R. concrete is the *amount* of finishing; that is, finishing manipulations should be held to a minimum. The scaling of the concrete in the runways at the Rome Air Depot and a similar type of scaling just reported* at Westover Field, Massachusetts, was due to excessive finishing which resulted in an accumulation of sand mortar at the surface. It is this mortar layer which is scaling off due to different coefficients of expansion of the superficial layer and of the concrete. Reduction of the quantity of sand in the mixture, or of the sand-aggregate ratio, will reduce the tendency toward the formation of the superficial mortar layer, but this precaution should be reinforced by a minimum of passes with the mechanical finisher and manipulations in the floating, belting and final trowelling at edges and joints. These precautions which apply also to plain cement concrete pavements, where "manipulation scaling" is relatively common, are of greater importance when Vinsol resin is used, because of the undersirable and persistent tendency toward excessive use of sand and lack of realization of the deceptively high plasticity of such mixtures.

(f) The rate of early strength gain of the concrete mixture is retarded by low temperature to a slightly greater extent than is usual for plain cement concrete. The retardation usually does not affect the time at which the finishing operation may and should be accomplished, but it may delay the time at which it is desired to remove forms. It is most noticeable in pavement slabs which were not protected from low temperatures after the finishing operation and when it is desired to re-use side-forms within 24 hours.

CLOSURE

The experience gained with concrete containing Vinsol resin in the construction discussed herein has led to the conviction of the writer that the entrainment of minute discrete air voids in concrete in amounts not greater than 7 per cent and not less than 3 per cent (optimum 4 to 5 per cent) by means of an air-entraining agent similar to Vinsol resin constitutes a major improvement in the manufacture of concrete. The tangible benefits during construction are: (1) materially increased plasticity and placeability, and (2) materially reduced bleeding and segregation of coarse aggregate. The benefit to the finished structure is a material increase in durability. The liabilities or hazards are the possibility of excessive entrainment of air and some slight reduction in strength.

Definite progress has been made in minimizing the hazard of excessive air-entrainment by the modification of the cement specification (A.S.T.M.

*Too late for full discussion herein.

Designation: C175) in 1944 to limit the air entrained in a test mortar in lieu of the previous specification, under which most of the construction discussed was built, which placed only a quantitative limit on the Vinsol resin present. Recent experience with cement manufactured under this specification has indicated that normal and reasonably predictable air contents were obtained. Additional insurance against excessive air is afforded by the determination of the air-entraining characteristic of the mixture in the design stage prior to construction, and repeated tests of unit weight (A.S.T.M. Method C 138-44) in the field during construction.

Reductions in strength have been and can be minimized or eliminated by proper reductions in water-cement ratio and sand content in the mixture design and in the field.

However, it must be understood that the practice of air-entrainment produces what is in effect a new type of concrete mixture—one with three aggregates (air, sand and gravel or stone) instead of the conventional two. If one of these aggregates (air) is inflexibly associated with the cement, and therefore non-regulable in the field, full control of the mixture is not available to the engineer. Although fully aware of the usual inconveniences and normally opposed to the use of admixtures in the field, the writer believes that the ultimate benefits potentially available in the practice of air-entrainment will not be realized until the air-entraining agent is treated as an aggregate and batched mechanically, by closely regulable means, in the field.

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Vol. 17 No. 1 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN September 1945

Job Problems and Practice

Five cash awards—$50.00, $25.00 and 3 of $10.00 each are to be made for the best contributions to this department in the current volume year—Sept. 1945 to June 1946.

In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved —in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

Non-skid Concrete Surfacing for Wooden Trestles at Mud Mountain Dam (42-168)

By H. H. ROBERTS*

A concrete-surfaced wooden trestle provided an unusual means of trucking in materials for Mud Mountain Dam, approximately 40 miles south and east of Seattle, Wash. on the White River. It is a rock fill dam and was constructed by the Guy F. Atkinson Co. of San Francisco, for the U. S. Engineers during 1939 through 1941. The purpose of this structure is for flood control, no hydro-electric facilities being provided.

Quarry sites were approximately 4½ miles down-stream from the dam site. Suitable haul roads were built but about 5200 linear feet had to be traversed through sections where standard roadways could not be built, necessitating the use of timber trestles.

Twenty-cubic yard trucks were chosen for the rock haul and the question of skid-proofing the deck and the trestles became an important problem. At first a bituminous surface treatment was considered, but because of the deflection in the deck system, this was rejected. Concrete,

*Chief Engineer, Contractors Pacific Naval Air Bases, Port Hueneme, Calif.

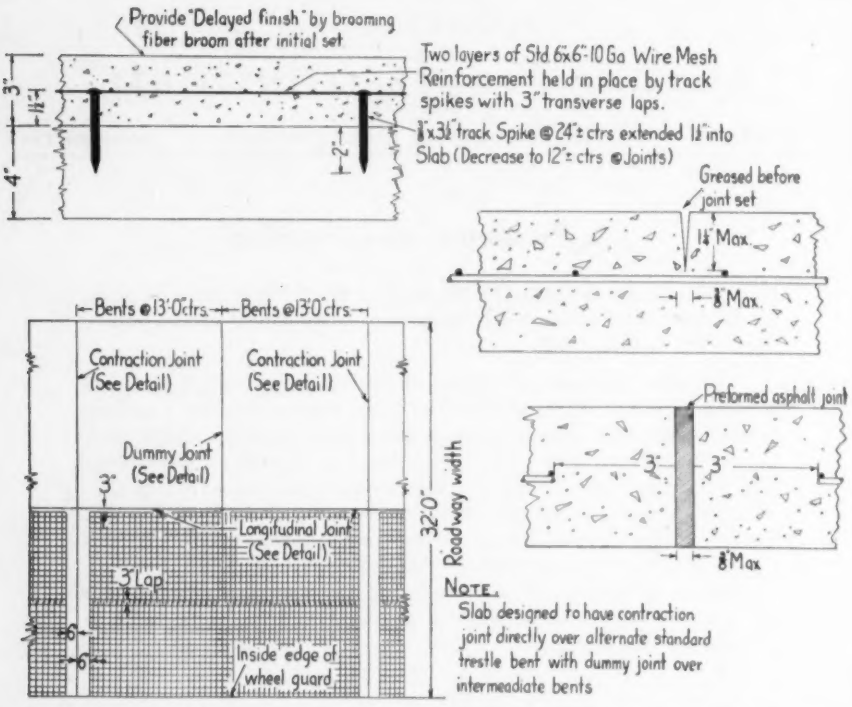
Provide "Delayed finish" by brooming fiber broom after initial set
Two layers of Std. 6"x6"-10 Ga. Wire Mesh Reinforcement held in place by track spikes with 3" transverse laps.
1"x3½" track Spike @ 24"± ctrs extended 1½" into Slab (Decrease to 12"± ctrs @ Joints)
3"
1½"
2"
4"
Greased before joint set
¼" Max.
⅛" Max.
Bents @ 13'-0" ctrs.
Bents @ 13'-0" ctrs.
Contraction Joint (See Detail)
Contraction Joint (See Detail)
Dummy Joint (See Detail)
3"
Longitudinal Joint (See Detail)
3' Lap
6"
6"
Inside edge of wheel guard
32'-0" Roadway width
Preformed asphalt joint
3"
3"
⅛" Max.
NOTE.
Slab designed to have contraction joint directly over alternate standard trestle bent with dummy joint over intermeadiate bents

Fig. 1—Three inch concrete deck slab on wooden trestle. The slab section is at upper left; deck plan, standard panel 16 by 26 ft., lower left; top right, detail of typical dummy joint; and typical transverse and longitudinal contraction joint at lower right.

therefore, became our next consideration, and after thorough investigation it was decided that a concrete wearing course 3 in. thick would be built. (Note details Fig. 1).

It was observed that prior to placing the concrete course the deflection at mid-span under a 55 ton load (20 cu. yd. truck loaded) was from 1¼ to 1½ in. After the surfacing was added this was reduced to ¼ to ⅜ in., the concrete developing the full continuity of the deck members. More than 1,000,000 cu. yd. of quarry rock was hauled over these trestles during a six month period and no appreciable damage to the concrete surface nor the deck system was noted. It is true that considerable cracking occurred, but there was little or no concrete removed by the traffic. As to safety, it is interesting to note that there were no accidents caused by skidding during the complete hauling operation.

The cost of the concrete deck system was high but the no-maintenance feature, and the high-speed traffic in wet weather (approx. 110 in. average rainfall per year) justified the cost.

Two types of trestle construction were used: (1) the single lane having a roadway width of 15 ft.-0 in. in the clear and (2) the double lane having a roadway width of 32 ft.-0 in. in the clear. The single lane is supported by a 5-pile bent and the double by a 9-pile bent, all piles being driven to adequate bearing and satisfactory refusal.

Both deck systems were identical, differing only in width. The members used were: 12 x 14 in. caps, 8 x 18 in. stringers (solidly bridged), 3 x 10 in. bracing, 4 x 12 in. decking.

All trestles were surfaced with a 3 in. reinforced concrete slab placed in 26 ft. panels, the panel points and longitudinal joints being provided with a preformed expansion joint filler and dummy joints being formed at the midpoints of the panels. 12 x 12 in. wheel guards were used on both sides of the double and on the outside edge of the single trestles. A 3 ft. walkway on one side only completed the structure. On the walkway side and outside the railing 50 gal. fire barrels and ladders leading to the ground were provided for fire protection. These were placed at approximately 150 ft. centers.

STRUCTURAL QUANTITIES

Unit	Length lin. ft.	Timber f.b.m.	Piling lin. ft.	Wire mesh sq. ft.	Spikes lb.	Exp. jt. filler lin. ft.	Concrete cu. yd
Single Trestle	1912	619,488	25,812	57,360	3,000	3,022	266
Double Trestle	3134	1,711,164	76,783	200,576	8,500	7,134	928
Quarry Road Bridges	158	84,688	5,530				
TOTAL	5204	2,415,340*	108,125	257,936	11,500	10,156	1194

*In addition to the above, approx. 91,250 lb. misc. hardware, including drift pins, bolts, washers and boat spikes were used.

This is a summary of cost of the trestle slab concrete:

Single trestle: $\frac{1912 \text{ ft. x } 15^5 \text{ ft.}}{9} = 3293$ sq. yd; \$6,324.25 or \$1.92 per sq. yd.

Double trestle: $\frac{3134 \text{ ft. x } 32 \text{ ft.}}{9} = 11{,}143$ sq. yd., \$27,229.50 or \$2.44 per sq. yd.

Cost per cu. yd. for material hauled: $27{,}229.50 + 6{,}324.25 = \frac{33{,}553.75}{1{,}000{,}000} =$ \$0.0335 per cu. yd.

A 6½ sack, 1: 2¼: 4¾ concrete at 1 in. slump and using a 1 in. maximum size aggregate was used and was thoroughly vibrated by means of a heavy bull float upon which two "wiggle-tail" air vibrators were

mounted. Following the bull float a wood trowel finish was added and all joints were made with a standard edging tool. Prior to initial set the surface was given a rough texture finish with a fiber broom, using a wavy longitudinal motion. This treatment provided an excellent non-skid finish. The surface was then cured, using Hunt process clear, and traffic was not admitted for 120 hours.

The concrete was all mixed in a central plant, transported to the site in dump trucks and dumped and hand shoveled into place. Close inspection during the mixing, placing and finishing resulted in an excellent job, this inspection being accomplished by contractors' forces.

It is the writer's opinion that this thin slab treatment can be used successfully for the surfacing of timber structures, thus insuring a safe wearing surface and almost entirely eliminating maintenance.

The writer designed the slab and supervised the construction while employed as assistant engineer for the Guy F. Atkinson Co., whose project manager was R. H. Northcutt; general superintendent, D. E. Root, and chief engineer, Donald O. Nelson.

Exposure of Concrete to High Temperature (42-169)

Q—I am desirous of obtaining some information regarding the behavior of concrete when exposed to high temperatures. I notice that building codes when specifying concrete for fire protection for steel beams, specify that it shall withstand temperatures varying from 1000 to 2500 or 3000 F. for periods ranging from one to four hours. I also understand that the Underwriters Laboratory have a requirement that concrete shall not be exposed continuously to temperatures greater than 500 F.

What I should like to get is a curve or charts showing the temperatures which concrete will withstand successfully in that interval between a few hours and continuous exposure.

A—While we are unable to find the kind of information you want, we have a letter from a competent ACI Member authority to this effect:

We know of no systematic and comprehensive investigation of the subject, and the available information is meager and that from different sources is somewhat contradictory. See Proceedings of the American Concrete Institute: V. 35, p. 292 and 417 (1939), and V. 36, p. 216 (1940). It is well known that the effects of the exposure of concrete to high temperatures depend significantly upon the kind of aggregate and the duration of the exposure. Concretes containing siliceous aggregates are not likely to lose strength rapidly if exposed to temperatures as high as 700 F, whereas those containing only aggregates of burned-clay, slag, cinder, pumice, limestone, and other non-siliceous materials lose strength slowly

at temperatures less than about 800 F. Available information seems to indicate that concretes containing the better types of aggregates for such exposures would be affected by prolonged heating somewhat as follows:

300 F—Only very minor loss of strength; 600 F, possibly 25 percent reduction of strength; 800 F, possibly 40 percent reduction of strength; 1000 F, possibly 50 percent reduction of strength, but may remain intact; 1500 F, ultimately disintegrates.

For extremely high temperature services, the high alumina cement Lumnite is preferred and is used in the refractory concrete of metallurgical and ceramic furnaces.

Derrick Stone and Cobbles in Mass Concrete (42-170)

Q—To questions on the use of derrick stone and cobbles and test cylinders of concrete containing larger aggregate an ACI Member writes:

A—There is seldom any advantage in using derrick stone in mass concrete when cobbles are available which may be incorporated in the mix. The derrick stone requires a duplicate facility for handling only 20 percent of the mass which can usually be secured considerably cheaper by simply mixing and placing that much more concrete. Further, when derrick stones are used, wetter and richer concrete using smaller than 6-in. maximum size aggregate should be used to secure good embedment. Concrete containing 6-in. cobbles has been successfully mixed in a 1-yard paving mixer and probably 4½-inch maximum concrete could be mixed in a ½-yard mixer. However, unless the mixers of the tilting type are used there will be some difficulty discharging this type of concrete as dry as it should be for best results as mass concrete (1½-in. slump with good vibration). Also because batch to batch variations are greater for small batches, the largest practicable mixer is preferable.

Aggregate in test specimens should not be larger than one-third the diameter of the specimen and preferably not larger than one-fourth. As this results in 18 to 24 in. diameter cylinders for 6-in. maximum concrete few specimens of this size are made. Usually 6-in. diameter cylinders are made, sometimes 8-in., and aggregate larger than 2 or 2½ in. is screened or picked out of the concrete making up the specimen. The height of standard specimens is twice the diameter; however, considerable labor is saved on larger diameter cylinders having height only 1.5 times the diameter and results are the same when multiplied by 0.98. There is considerable conflict in data on the factor to apply to small specimens to secure the true strength of mass concrete. Probably the safest procedure is to be guided by results which indicate the mass strength to be somewhat less than that of 6-in cylinders, possibly by as much as 10 to 20 percent. See ACI *Proceedings*, 1935, p. 280.

Based on an average water requirement of 205 lb. per yd. of 6-in. maximum rounded gravel concrete and 220 for similar 4½-in. maximum con-

crete of 1½-in. slump, and a W/C of 0.65 for both, cement requirements would be 315 and 339 lb. per yd., respectively. Probably the same difference of 15 lb. of water per yd. would apply to other types of aggregate so that the estimated difference in cement content for the two maximum sizes would vary only with the water-cement ratio; the smaller the ratio, the greater would be the difference in cement content.

In some of these and related matters the Concrete Manual of the Bureau of Reclamation, fourth edition, may be of assistance.

Effect of Brine on Concrete (42-171)

Q—To "what might be the effects of concrete exposure to brine," we have this from an ACI Member.

A—It is not considered that dense, high quality concrete will be unfavorably affected by brine unless alternate soaking and drying occurs. The concrete should have a minimum of six sacks of cement per cubic yard of concrete and the water cement ratio, including the free or surface water in the aggregate, should not exceed 6 gal. per sack of cement (W/C = 0.53 by weight). If the fine aggregate available is lacking in fines it may be desirable to add a workability agent such as an air-entraining admixture to insure concrete of excellent workability without the use of excessive mixing water.

If the concrete will be alternately wet with the brine, and dried, some further protection should be provided. A 2-coat treatment of linseed oil is recommended in this case. The concrete should be seasoned and dried before the application of the linseed oil, or else treated with magnesium fluosilicate solution (2 lb. crystals per gallon of water) to "kill" the lime.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Effect of type of test specimen on apparent compressive strength of concrete

BRYANT MATHER; ASTM 1945 Preprint. Synopsis by AUTHOR

Two programs of strength testing are reported, in each of which concrete from the same sample or batch was tested using either drilled cores or molded cylinders for comparison with modified cubes. A total of 2375 strength tests were made and from the resulting data it is suggested that the choice of type of specimen may considerably affect the compressive strength value obtained for the concrete.

Stone dust in aggregate

ARNOLD ARNSTEIN, *Concrete and Constructional Engineering*, V. 40, No. 3, (March 1945) pp. 43-44 Reviewed by GLENN MURPHY

The author reports tests of concrete made by the Public Works Department in Tel-Aviv. Although in some of the mixtures the material passing the No. 200 sieve was as much as 27 per cent (by weight) of the fine aggregate (greatly exceeding the 3 per cent limit given in the D. S. I. R. Code of Practice), all but two samples met the strength requirement. The water-cement ratios varied from 0.75 to 1.00 (by weight). The author concludes that the dust may be useful in decreasing harshness of mixtures.

Automatic accelerated freezing and thawing apparatus for concrete

CHARLES E. WUERPEL AND HERBERT K. COOK; ASTM 1945 Preprint. Synopsis by AUTHOR

A detailed structural and operational description is given of a newly developed apparatus for automatically and rapidly freezing and thawing 3½ by 4½ by 16-in. concrete specimens. The temperature of the capacity load of 102 specimens is reversed from 42 F. to 0 F. and from 0 F. to 42 F. in a 2-hr. cycle. Typical results of tests to determine the influence of aggregate type on the durability of concrete made with and without an air-entraining admixture are presented to illustrate the practicability of using the apparatus for acceptance testing based on freezing and thawing.

Portland cement dispersion by adsorption of calcium lignosulfonate

FRED M. ERNSBERGER and WESLEY G. FRANCE. *Industrial and Engineering Chemistry*, Industrial Edition. V. 37, No. 6, pp. 598-600, June 1945. Reviewed by R. N. YOUNG

Turbidity tests utilizing the Wagner turbidimeter were conducted on suspensions of portland cement in water, with and without additions of calcium lignosulfonate. Apparent increase in specific surface with addition of the dispersing agent reached maximum of 48 per cent. Determination of adsorption isotherm of calcium lignosulfonate

by cement in water suspension is described, data are given for a typical cement. Experiments employing electrophoresis cell are described. Authors conclude: That the lignin salt is a colloidal electrolyte, that anions of lignin salt are positively absorbed by cement particles, that increase in dispersion of cement is a function of adsorbed dispersing agent.

Studies of concrete with entrained air

DELMAR L. BLOEM and STANTON WALKER, Technical Information Letter 22, 1945, of National Ready Mixed Concrete Association. Reviewed by AUTHORS

Progress report on preliminary group of concrete tests carried out in Research Laboratory of National Ready Mixed Concrete Association at University of Maryland. Principal purpose to obtain information on effect of entrained air on mixing water requirements for concrete. To magnify results large quantities of air (about 10 percent) entrained. Strength and "dynamic E" tests of 6 by 12-in. cylinders also made. Amount of air measured for hardened concrete as well as fresh concrete. Data indicate: (1) Mixing water can be reduced from about 45 percent (lean mixes) to 20 percent (rich mixes) of volume total air entrained; (2) Approximately 30 percent more air in fresh concrete than in hardened concrete; (3) Less reduction in strength due to air than to equivalent volume of mixing water.

Effect of length on the strength of compression test specimens

J. TUCKER, Jr.; ASTM 1945 Preprint. Synopsis by AUTHOR

When compression test specimens of concrete are shorter than twice their least width it is customary to adjust the test values by the use of a single set of correction factors, such as those in Standard Methods of Securing, Preparing, and Testing Specimens from Hardened Concrete for Compressive and Flexural Strengths (C 42-44), the corrections depending only upon the ratio of length to least width. This paper demonstrates that the extent of the variation in strength with length is dependent upon several distinct factors. For example, the relation between strength and slenderness of specimen varies with the strength of the concrete and therefore the use of a single set of correction factors is not warranted. Much more reliable estimates of strength may be obtained by limiting the range of slenderness values to those for which the correction is small, even though this may require diameters less than those usually considered acceptable. Although the numerical values which are given apply to concrete, the principles are applicable to other brittle materials.

Equations for computing elastic constants from flexural and torsional resonant frequencies of vibration of prisms and cylinders

GERALD PICKETT; ASTM 1945 Preprint Synopsis by AUTHOR

The equations of Mason, Love, etc., that do not take into account the effects of shear are not so satisfactory as Timoshenko's differential equation for flexural (transverse) vibration of prismatic bars. Goens' solution of Timoshenko's differential equation for bars with free ends is in very close agreement with results obtained by means of the mathematical theory of elasticity if the shear constant K' is given the proper value. The proper value depends on Poisson's ratio, being about $\frac{9}{8}$ for $\mu = 0$, $\frac{8}{9}$ for $\mu = \frac{1}{6}$, and 0.85 for $\mu = \frac{1}{3}$. The radius of gyration r adequately describes the cross-section of a prism for the computation of resonant flexural vibration. The approximate equation given by Goens for the correction factor T_1 can be made accurate to at least three figures by subtracting the quantity $125(r/l)^4$.

Equations are given for computing Young's modulus and the shear modulus from flexural and torsional resonant frequencies, respectively.

Graphs are given from which the size and shape factor C in the equation for Young's modulus and Goens' correction factor T are readily determined.

An equation is given for obtaining Poisson's ratio from the flexural and torsional resonant frequencies, and the limitations of the equation are discussed.

Bridge foundations

W. A. Fairhurst, *Concrete and Constructional Engineering*, V. 40, No. 3, (Mar. 1945) pp. 37-43
Reviewed by Glenn Murphy

This is the concluding installment of a series dealing with design and construction features of bridge foundations. The use of counterweighted abutments is shown in the description of the Dinnet Bridge over the Dee River. The bridge over Grange Burn, Sterlingshire, illustrates the use of deep cellular foundations so proportioned that the structure develops the same load on the subgrade as did the excavated material. This construction is advantageous where the allowable bearing pressure is abnormally low.

Preliminary boring for the foundations of the Kerse bridge showed 5 ft. of mud and stone, 3 ft. of sand and gravel, and at least 83 ft. of soft silty clay. To meet this condition a collar pile was devised. A 3½ ft. square collar, 18 in. thick, was laid in the mud and the lower end of a 35-ft. tapered reinforced concrete pile started through the hole in the collar. Each pile was constructed with a bulge near the top. The pile was driven until the bulge contacted the collar, then both the pile and the collar were driven down. A proof load of 90 tons was placed on one of the piles, resulting in an total settlement of less than ½ in. in 2 weeks. No further deflection occurred when the load was reduced to 75 tons. The design load was 22 tons. The author reports the bridge to be in perfect condition after 12 years service.

Rings in cement kilns, part I: why they form and how to prevent them

Harold R. Gingerich, *Rock Products*, V. 48, No. 6, pp. 89-90, June 1945. Reviewed by R. N. Young

Articles deal with formation of slurry rings and clinker rings in cement kilns, effects of kiln design and operation, methods of removing and preventing. Part I deals with influence of materials and "Laboratory Control" on clinker ring formation. Physical and chemical properties of raw materials are said to influence kiln deposits. Refractory lining is important. Fusion of clinker with kiln lining promotes ring formation. Uneven surface of refractory provides a convenient point for start of ring formation. Viscosity of the molten cement materials is a factor. A mobile liquid with high alkali content will usually develop ring formation readily. High ash coal, with low melting point of the ash promotes rings in area where coal impinges on load, apt to occur when composition of ash varies. Content of *Fe*, *Al*, *S* in ash is a leading factor, as is also fineness of coal, coarse ground fuel tending to aid in ring formation. Under laboratory controls, some factors listed as promoting formation of rings are: uneven composition of raw mix, low *Si-Fe*, *Al* ratio, high *Al* (by agglomeration of clinker), high *Fe*, reducing atmosphere in kiln, coarse raw fineness (except when kiln burning temperatures are high). Range in clinkering temperature depends on change in liquid content of clinker with change in temperature. Range may be wide or narrow. It is thought that ring formation is less apt to occur when the range is wide.

Theory of elastically restrained beams, applied to statically indeterminate reinforced concrete beams

M. Hilal (Lecturer, Faculty of Engineering, Giza). Costa Tsoumas & Co. Press, 1945.
Reviewed by F. E. Richart

In this booklet the author presents solutions of a number of problems in continuous beams and frames, using well known methods of analysis and notation commonly

used in Europe. While the title implies that the treatment applies particularly to reinforced concrete, the analysis applies to elastic members of any material, and little attention is given to details of design or analysis peculiar to reinforced concrete.

The method of "fixed points" is used throughout, and analytical and graphical determinations of the position of such points are given for members of constant and varying moments of inertia.

Solutions are given for continuous beams on free supports, including tabulated values for influence lines for moments and shears in beams of 2, 3 and 4 equal spans. Graphical solutions are also demonstrated. Maximum and minimum moments and shears due to live loads, and the effect of settlement of supports are treated.

For certain continuous frames analytical and graphical determinations of the fixed points are shown. Applications are made to beams supported on girders which provide torsional restraint; also to the determination of moments due to temperature and shrinkage and to movements of the supports. For continuous multiple frames, the "equation of four moments" and the equation of deformation angles (essentially the "slope—deflection" equations) are stated.

While it probably contains little that is new to students of the theory of continuous frames, this booklet forms a convenient reference to the "fixed point" method of analysis, particularly to American engineers since it is written in English.

The proportioning of concrete (dosagem de concretos)

F. L. L. Carneiro, Instituto Nacional de Tecnologia, Rio de Janeiro, 1943. *Road Abstracts*, V. XII, No. 5, May 1, 1945. Highway Research Abstracts

The formulas of D. Abrams, J. Bolomey, O. Graf, I. Lyse, and other workers, correlating the compressive strength of concrete with the water-cemet ratio, are critically reviewed, and the method of proportioning is discussed that has been used since 1937, by the National Institute of Technology of Brazil. The formula employed for proportioning relates the water-cement ratio with the total weight of aggregate per unit weight of cement and the weight of water required expressed as a percentage of the total weight of dry materials. The properties and functions of cement, aggregates and water are reviewed in their relation to the quality of concrete, and Brazilian standard specifications for different types of cement and for the sampling, grading and testing of aggregates are quoted in full. Methods are discussed of pre-determining properties of concrete such as compressive and tensile strengths and impermeability and durability, and the relevant clauses are quoted of the Brazilian standards for the strength of structural reinforced concrete and for preparing and curing cylindrical test specimens. Grading and proportioning of aggregates, and workability of concrete, are examined. The grading and proportioning charts used by the National Institute of Technology are reproduced, and the percentages of water required with different sizes of coarse aggregate are tabulated for concrete to be compacted (a) by manual methods, and (b) by vibration. Recommendations are made regarding control and sampling on the site, and permissible tolerances are indicated. An appendix describes briefly a method of determining the tensile strength of test cylinders.

Experiments with admixtures for reducing the mixing water content and improving the workability of concrete

W. Kronsbein, *Zement*, 1943, 32 (19/20), 209-17. *Building Science Abstracts*, V. XVII (New Series) No. 10, Oct. 1944. Highway Research Abstracts

A brief review is made of the action of admixtures to improve the workability and strength of concrete and of published results of tests of various proprietary products. This is followed by an account of experiments with two proprietary admixtures, "Beton-

plast" and "Murasit" in concrete made with two portland cements and one portland-blastfurnace cement. The reduction of the water content for the same workability of the mix varied, e.g., it was in some cases about 12 per cent, in others less than 8 per cent. An increase of compressive strength of 30 per cent or more, representing a cement economy of about 70 per cent, was obtained in one case, in others the effect in strength was slight. The degree of water-tightness obtained by the admixtures also varied. In general, the experiments showed the effects of the admixtures on the concrete properties to be by no means the same for all concretes. The reduction of the quantity of mixing water made possible differed greatly for different cements and did not always mean an economy of cement. In some cases, the concrete from the mix with a reduced water content had not a denser structure but was more or less porous and there was no increase but even a reduction of strength. Where, in spite of its porous structure, the concrete showed an improvement in water-tightness, this was attributable to the formation of closed pores due to the admixture. The differing results obtained with the admixtures would seem to be largely due to difference in the cement properties; with the few cements used for the experiments the results obtained were very marked. It is to be assumed that the particle structure of the mix, the aggregate shape and grading and the absolute water content also influence the result. In general, the use of admixtures to obtain a definite effect would not seem possible. In every case it would be necessary to carry out preliminary experiments with an admixture in order to determine whether advantages are gained by its use, e.g., a reduction of the mixing water content allowing an economy of cement, an improvement in workability, the prevention of segregation.

Stress conditions for the failure of saturated concrete and rock

K. Terzaghi, ASTM 1945 Preprint — Reviewed by Gerald Pickett

An analysis is made of the stress conditions leading to failure of porous bodies that are under compression. Previous data are examined and conclusions are drawn. Three types of failure are described: (a) splitting, which is regarded as a failure in tension; (b) pseudo-shear, which is regarded as a failure partly in tension and partly in shear; (c) shear. The equation given by Ros for increase in strength with increase in confining pressure is considered to be applicable to materials that fail by splitting. The equation is

$$q_c' = q_u + p_c \frac{q_u}{q_t}$$

where q_c is the compressive strength when the confining pressure is p_c
q_u and q_t are the compressive and tensile strengths, respectively, under uniaxial loading.

Terzaghi gives a more complicated equation containing functions of an unknown angle for materials like concrete that fail in pseudo-shear. For such materials, he explains the non-linear increase in strength with increase in confining pressure as being due to a greater tendency to fail in shear when a confining pressure is applied, the greater the confining pressure, the greater the tendency to fail in shear. The equations just mentioned apply when the pores are either empty or, if filled with liquid, are under no hydrostatic pressure. Terzaghi explains that if the liquid penetrates the pores, the confining hydrostatic pressure is not fully effective. He next describes a test in which the pores are completely filled with liquid, the liquid being under a hydrostatic pressure equal to the confining pressure. He mentions previously conducted tests of this kind on concrete in which there was very little increase in strength with increase in confining pressure. From these results Terzaghi reasons that the individual constituents of

saturated concrete are almost entirely surrounded by liquid. He further reasons that the voids must therefore consist of narrow slits.

Although it appears that Terzaghi is rather firmly convinced of the correctness of his analysis, he mentions that the basis for his analysis is semi-empirical. Hence, final answers must await the results of more conclusive tests. He describes the essential conditions for such tests.

Evaluating the mechanical strength of coarse aggregates

A. H. D. Markwick and F. A. Shergold. (Abstract of paper No. 5447, *Journal of the Institution of Civil Engineers*, Apr. 1945). *The Surveyor and Municipal and County Engineer*, V. CIV, No. 2782, May 18, 1945, London. Highway Research Abstracts

This paper relates to a test developed at the Road Research Laboratory to meet the need for a single test that can be quickly and simply carried out on material as crushed and supplied for use in construction, in which condition it is termed "aggregate".

In the standard aggregate crushing test a measured quantity of aggregate (about 3 kilogrammes) that has been sifted between ½-in. and ⅜-in. British Standard sieves and oven-dried for 24 hrs. at 100-110 deg. C. (212-230 deg. D.) is placed in a 6-in. diameter cylindrical steel mould with a close-fitting plunger, and subjected to a load increasing uniformly to 40 tons, in 10 min. The aggregate crushing value is determined by the percentage of fines passing a No. 7 British Standard sieve after test, and is an inverse measure of the mechanical strength of the aggregate. Values range from 5.3 for an exceptionally hard hornfels to 72.8 for foamed slag or soft brick. The values for hard igneous rocks, quartzites, flints and sandstones range from 10 to 20, for limestones from 20 to 30, and for slags from 25 upwards. In developing the test, factors affecting the results, such as size of sample, total load, rate of application, drying conditions and method of compaction were standardized. Particle shape was not standardized and the authors state that flaky and elongated particles gave somewhat poor test values.

To obtain geometrical similarity, when aggregates of various sizes are tested, the ratio of the aggregate size to the aperture size of the sieve used to separate the fines should, the authors explain, be kept constant, and the size of the test cylinder should also, in theory, be varied proportionately to the aggregate size, though in practice it has been found that three sizes of cylinder suffice to cover the usual range of aggregate sizes. Details of the conditions for testing various sizes of aggregates are shown in the following table:

RECOMMENDED TEST CONDITIONS FOR AGGREGATE CRUSHING TESTS ON VARIOUS SIZES OF AGGREGATES

	*Size of aggregate to be tested (B.S. sieves: inches)								
Passing Retained on	2 1½	1½ 1	1 ¾	¾ ½	½ ⅜	⅜ ¼	¼ ⅛	⅛ ⅛	⅛ No 7
Size of test sieve for separating fines: inches	⅝	¼	⅛	¼	No. 7	No. 10	No. 14	No. 18	No. 25
Diameter of test cylinder: inches	12	12	12	6	6	6	3	3	3
Load applied: tons	160	160	160	40	40	40	10	10	10
Relative volume of test sample	8	8	8	1	1	1	⅛	⅛	⅛

*The larger size of sieve defines the nominal size of the aggregate.

The effect of size on crushing strength was illustrated by the result of a test on cylindrical specimens of a limestone the strength falling from 29,000 psi. for 1-in. (diameter and height) specimens to 20,000 psi. for 2-in. specimens.

As regards correlation with other tests, on the same materials, the crushing values of a large number of ½-in. aggregates showed: (1) Almost numerical agreement with the Los Angeles values; (2) fair correlation with crushing strength and impact values; (3) poor correlation with dry, and wet attrition values; (4) little correlation with water absorption and abrasion values.

Disintegration of bridge concrete in the west

F. H. Jackson, *Public Roads*, V. 24, No. 4 Reviewed by the Author

This report gives the results of an inspection of approximately 200 concrete structures in areas of Wyoming, Oregon, Washington and California, in connection with a study of the causes of premature failure of concrete in certain portions of these states. The report also includes discussion of possible reasons for the failures and suggestions for revisions in specifications to govern future work.

Many of the older projects, particularly those built prior to 1930, were in surprisingly good condition. However, the concrete in many of the bridges, including a rather disturbingly large proportion of the newer structures, showed sufficient evidence of distress to warrant considerable concern.

During the course of these inspections, four distinct types of concrete deterioration were observed. These were classified in the report as follows:

Type 1.—Deterioration due to gradual or normal weathering. This is usually indicated by slight surface erosion, pitting, rounded corners, etc. Many old structures also show cracks due to settlement and impact from colliding vehicles. This condition is to be expected and is evidence of durable concrete, especially when found on older structures subject to severe weathering.

Type 2.—Deterioration due to accelerated weathering. This type of failure is unfortunately rather common, particularly in areas subjected to severe frost action. It is usually evidenced first by the formation of fine cracks or "D-lines" on the surfaces of exposed members. Concrete so affected has little strength and the matrix has a dull, chalky appearance in sharp contrast to the dense, compact bluish-gray matrix usually found in good concrete. Rapid and progressive disintegration of the concrete frequently follows the appearance of "D-lines."

Type 3.—Deterioration due to salt scaling. This type is the result of using calcium or sodium chloride for ice removal and will almost invariably occur on surfaces so treated, particularly if the concrete is new.

Type 4.—Cracking due to abnormal expansion. This type of failure is due to internal expansion of the concrete resulting in a formation of relatively wide, open cracks of appreciable depth in the surface of the member. The principal example of this type of failure is the expansion due to reaction between the alkalies in portland cement and the siliceous constituents of certain aggregates.

Evidence of type 2 deterioration was found in many structures in those areas of Wyoming, Oregon and Washington which are subject to severe weathering. In some cases there was evidence that weathering of the type 2 variety had been preceded by internal expansion (type 4), with final failure as the result of a combination of the two. However, in many places there was no evidence that alkali reaction had been a factor in the disintegration of the concrete; failure in these instances being clearly the result of accelerated natural weathering.

The report discusses the possibility of using air entrainment to increase the durability of concrete, as well as certain restrictions on the alkali content of portland cement and the necessity for examination of the alkali-reactive characteristics of aggregates in

areas which seem to be subject to this action. The report also discusses the apparently superior performance of old-fashioned cements as compared to the portland cement now being manufactured. It concludes with the following recommendations:

That, where concrete is to be exposed to severe frost action, specifications be revised, where necessary, to require, in addition to a suitable cement content, that the free water content of the mix shall, in no case, exceed 6.0 gallons per sack of cement.

That, where concrete structures or portions of structures (including pavements) will be exposed to severe frost action, provision be made to entrain sufficient air in the fresh concrete so as to produce a total air content of from 3 to 5 percent, based on the theoretical weight of the air-free concrete.

That the desired air entrainment be obtained preferably by adding an approved air-entraining agent to the concrete at the time of mixing, in such quantity as will maintain the percentage of air within the limits specified.

That approval of any material proposed for use as an air-entraining agent be based on data, obtained from either research or field use, or both, that are sufficiently comprehensive to demonstrate to the satisfaction of the contracting agency that the proposed material, when used as required, will not seriously affect the strength or other essential properties of the concrete.

That, when the desired air entrainment is to be obtained by the use of an air-entraining cement: (1) The cement meet the requirements for air entrainment given in A.S.T.M. Specification C 175-44 T; and (2) the specifications authorize the engineer to require such changes in materials, proportions, or methods of mixing as may be necessary from time to time to maintain the percentage of air within the limits specified.

That, in all areas where tests or previous experience indicate that alkali-reactive aggregates may be encountered, specifications for portland cement be modified by requiring that the total percentage of sodium oxide plus 0.658 times the percentage of potassium oxide shall not exceed 0.60.

Tests on square twisted steel bars and their application as reinforcement of concrete

K. HAJNAL-KONYI. Reprint from *The Structural Engineer*, V. XXI, No. 9. (Sept. 1943) pp. 327-368; Discussion Vol. XXII, No. 2 and 3 (Feb., Mar. 1944) pp. 66-92, 114-118. Reviewed by AUTHOR

The tests comprised 35 beams reinforced with small size square bars (5g and ½ in.) both twisted and untwisted. All beams were 6 in. wide and 7 ft.0 in. long, the depths varying between 4 and 10 in. Some were made of ordinary grade concrete (1:2:4 mix), some in high grade concrete (1:1:2 mix) and the amount of reinforcement varied between .059 percent and 1.785 percent for untwisted steel, and between .214 percent and 1.19 percent for twisted steel. The beams were tested on a span of 6 ft. 0 in. with two knife-edge loads at the third points. All beams had sufficient shear reinforcement to exclude failure by diagonal tension.

In the majority of the beams reinforced with twisted steel, up to .9 percent failure occurred by fracture of the reinforcement. This did not happen with mild steel even at $p = .214$ percent and, for this reason, two beams were made with .059 and .137 percent respectively, in both of which the mild steel bars could be fractured. In both beams the maximum load was reached at the first crack after which the load suddenly dropped and then increased again to a second maximum at which fracture of the reinforcement occurred. By this stage a very great deflection had occurred. The second maximum was lower than the first.

In all beams reinforced with twisted bars the maximum bending moment was substantially in excess of the value calculated from the ultimate strength of the reinforcement on the basis of the plastic theory. On the basis of the standard method the excess

was even greater in most of the beams. To explore this phenomenon special tensile tests were arranged. 5g bars both twisted and untwisted were fractured in concrete specimens of 3x3 in. at the critical section. Three tests with twisted bars agreed very well with each other, their average ultimate strength was 16 percent in excess of the ultimate strength of specimens cut from the same rods, when tested in the air. The untwisted bars showed no excess since the bond was destroyed over the full embedded length of the bars long before failure.

It appears that steel embedded in concrete shows an ultimate strength higher than that obtained in the usual test if the bond in the neighborhood of the crack is maintained on each side until failure. This phenomenon is similar to the well known notch effect and does not occur with steel having a definite yield point. It can be achieved with work hardened steel which does not yield.

Nothing in the behavior of the beams suggested that the conventional "yield" point of twisted steel (which is the stress corresponding to an elongation of .5 percent) was in any way significant. The tests have shown that the ultimate strength of twisted bars is a safe basis for calculating the ultimate bending moments.

In all tests the development of cracks was carefully observed. According to continental tests, a maximum width of .01 in. can be considered harmless from the point of view of corrosion. In no beam was this limit reached at a calculated stress lower than 40,000 psi.

The article points out deficiencies in the standard method of design and demonstrates that a far better agreement with test results can be obtained by the plastic theory (Whitney's method). It deals particularly with the reduction of the design factor M/bd^2 at the so called "economic" percentage, if the permissible steel stress is increased. This is a consequence of the standard method and handicaps the economic use of high tensile steel. The tests have proved that square twisted steel can safely be substituted for mild steel in the inverse ratio of its ultimate strength to the yield point of mild steel without increasing the cross section of the concrete. If the permissible stress for square twisted steel is fixed at 50 percent above that of mild steel, the factor of safety of beams reinforced with square twisted steel is higher than if mild steel of 1.5-timed the sectional area of the twisted steel had been used.

The permissible steel stress is limited by the width of cracks. On the assumption of a permissible width of .01 in. a permissible stress of 30,000 psi. would be safe for square twisted steel having an ultimate stress of not less than 75,000 psi. This applies to slabs and rectangular beams up to .9 percent in 1:2:4 concrete (min. cube strength 2850 psi.) and up to 1.2 percent in 1:1:2 concrete (min. cube strength 3750 psi.), reinforced with square twisted bars of a size not exceeding ½ in.

The article also deals with the loss of the high tensile properties of cold worked steel by the influence of heat. Tests have proved that permanent loss of high tensile properties does not occur if the temperature of the steel has not exceeded 400 C and is insignificant even if the steel has been heated to 500 C. On the other hand, once the temperature of mild steel has exceeded 500 C, the building is likely to collapse or at least to be beyond repair.

In the discussion the author, referring to an article by A. C. Vivian*, drew attention to the gross errors which were introduced in the "nominal" stress-strain diagram of a metal which underwent much change in area before fracture. Work hardening affects the volume of metal involved in the fracture, since the reduction of the sectional area

*Squadron-Leader A. C. Vivian: Mechanical Properties of Metals. *Engineering*. July 23 and Aug. 8, 1943, pp. 78-81 and 118-120. Paper entitled "A Renaissance of Mechanical Properties" presented to the Institution of Mechanical Engineers.

was considerably less than in the case of mild steel.* When the stress of both is mentioned it is to the nominal area and not to the actual area at fracture that is referred to. It may also explain the further increase in the ultimate strength if a bar is fractured while firmly gripped in concrete and thus the loss of the sectional area at fracture is further reduced.

Structural efficiency of transverse weakened-plane joints

EARL C. SUTHERLAND AND HARRY D. CASHELL, *Public Roads*, April-May-June 1945, V. 24, No. 4
Reviewed by the AUTHORS

Early in 1940 the Public Roads Administration sponsored a comprehensive investigation, in cooperation with several State Highway Departments, for the primary purpose of studying the effects of varying the spacing of expansion joints in pavements with closely spaced weakened-plane contraction joints. As a part of this investigation the Public Roads Administration carried out a program of special tests to determine the ability of joints of the weakened-plane type to reduce the critical stresses caused by loads acting near the joints.

In this study information was obtained on the structural behavior of weakened-plane joints as affected by: (1) type of coarse aggregate, (2) maximum size of coarse aggregate, (3) presence or absence of dowel bars, (4) method of producing fracture at the joint, (5) compressive forces acting to close the joint, and (6) width of joint opening.

The pavement on which these tests were made was divided into 6 sections, each 30 ft. long, 20 ft. wide and of 8-in. uniform thickness. Each section was definitely separated from those adjoining it and was divided longitudinally by a deformed metal center joint having ⅝-in. diameter tie bars at 60-in. intervals and divided transversely by a weakened-plane joint. Provision was made for opening and closing the joints, by mechanical means, so that they could be tested under compression and at various controlled openings.

To investigate the influence of the type and size of coarse aggregate, on the structural efficiency of weakened-plane joints, different concrete mixtures were used in the various sections. Potomac river gravel was used as the coarse aggregate in 4 sections, the maximum size being 1 in. in 2 sections and 2½ in., in the other two. The coarse aggregate in the two remaining sections was a crushed limestone obtained from near Frederick, Md. The maximum size of the aggregate was 1 in., in one of the limestone sections and 2½ in. in the other.

Load transfer devices were installed in one of the joints for comparing the relative efficiency of weakened-plane joints with and without load transfer devices. These consisted of ¾- by 24-in. plain round dowels spaced at intervals of 12 in.

The testing schedule was essentially the same for each of the joints. Briefly, this schedule consisted of:

1. Measurement of the critical strains at the interiors of the four panels of a given section for a load of selected magnitude.

2. With the joints under compression, caused by a horizontal force of 120,000 lb. acting longitudinally at one end of the section, the measurement of the critical strains caused by loads acting at the joint edges and corners.

3. Release of the compressive force and measurement of the critical strains caused by loads acting at the joint edges and corners for successive controlled joint openings of 0.037, 0.055, 0.073 and 0.092 inch.

*This may explain why the ultimate strength of work hardened steel is higher than that of mild steel.

4. Opening of the joint to a width of ½ in. and measurement of the critical strains caused by loads acting at the joint edges and corners. For this opening all effects of aggregate interlocking had disappeared and the slab edges at the joint were acting as free ends. In the case of the doweled joint all dowels were cut before testing at this opening.

5. Remeasurement of the critical strains caused by loads acting at the interior of the panels.

From the data obtained in these tests it was possible to compute the efficiency of the various joints in reducing the critical stresses caused by loads acting at the joint edges and corners. It was found that aggregate interlock was effective in stress control, both at the joint edges and joint corners, when the joints were under compression. At the 0.037-in. opening aggregate interlock was very effective in controlling the critical corner stresses, but it was found to be an uncertain means of stress control at the joint edges with this opening. At the 0.073-in. opening aggregate interlock was ineffective in stress control both at the joint edges and corners. The dowels definitely increased the effectiveness of the joint in reducing the critical corner stresses and improved the uniformity of stress reduction between the different corners and from point to point along the joint edge. However, the doweled weakened-plane joint was not highly effective in reducing the critical edge stresses probably because the diameter of the dowels was not sufficient to be effective in slabs of 8 in. or more in thickness.

The weakened-plane joints in the slabs with gravel aggregates were more effective than those in slabs with crushed stone aggregates. Also large gravel aggregate gave more effective stress control than small gravel aggregate, but the size of the aggregate had little influence on the effectiveness of the joints in the slabs with crushed-stone aggregate.

To determine the cause of this difference in the effectiveness of the joints with the different types of coarse aggregates, a portion of one of the panels forming the joint was removed from each section so that the faces of the joints could be examined. The examination showed that in the sections having gravel coarse aggregate the joint faces were rougher than those of sections containing crushed-stone coarse aggregate due to the fact that when the crack occurred at the joints, the crushed-stone aggregate fractured whereas the gravel aggregate, in most instances, pulled out through loss of bond. It should be recognized that different aggregates might act differently in this respect. That is, some gravel aggregates might fracture at the joints, giving a smooth face, while some crushed-stone aggregates might pull out.

It is concluded from this investigation that aggregate interlock is not a dependable method of stress control at weakened-plane joints. However, it has been found from field observations that aggregate interlock is helpful in preventing or reducing the amount of faulting at weakened-plane joints and cracks when they are not allowed to open too widely. In pavements laid with expansion joints there is a tendency for the contraction joints and cracks to open progressively until all available expansion space is dissipated and if the available expansion space is sufficient the joints and cracks may open so widely that the effectiveness of aggregate interlock, in this respect, will be destroyed. Thus, if the effectiveness of aggregate interlock for preventing or reducing faulting at weakened-plane joints and cracks is to be maintained it appears to be necessary to limit the amount of available expansion space in the pavement. This may be done either by actually eliminating expansion joints or by using an expansion joint filler that offers considerable resistance to compression.

ACI NEWS LETTER

Vol. 17 No. 1 JOURNAL of the AMERICAN CONCRETE INSTITUTE September 1945

REPORT OF 1945 NOMINATING COMMITTEE

The 1945 ACI Nominating Committee, Roderick B. Young, chairman, Roy W. Crum, Raymond E. Davis, Frank H. Jackson, J. C. Pearson, T. C. Powers, T. E. Stanton, Morton O. Withey report the following nominations for offices whose terms expire at the 1946 convention:

President

HARRISON F. GONNERMAN, Manager Research Laboratory, Portland Cement Association, Chicago, an ACI Member since 1918; Wason Research Medalist, Member Board of Direction as Director Fifth District since February 1943; elected Vice-President February 1945. A further record of his Institute activity will be found under his name in the ACI Directory. He is a member of the executive group of ACI Committee 115, Research; a member of the Publications Committee and in papers and otherwise has contributed much to the Institute's technical work.

Vice-President (to succeed himself)

STANTON WALKER, Engineering Director, National Sand & Gravel Association and of the National Ready Mixed Concrete Association, Washington, D. C.; an ACI Member since 1921; member of Standards Committee since 1937, and of the Advisory Committee, as Chairman Department 200, General Properties, since 1938 and became Chairman of the Advisory Committee, February 1945. Member Board of Direction since 1940 when he became Director Fourth District by appointment to fill a vacancy and twice elected; elected Director-at-Large for a three-year term, 1943 and Vice-President, 1945.

Vice-President

ROBERT F. BLANKS, Chief, Engineering and Geological Control and Research, Bureau of Reclamation, Denver, Colo.; ACI Member since 1932; member Publications Committee since 1941; Chairman since February 1945; Member Board of Direction as Director Sixth District, 1941-42; appointed by Board March 1944 to be Director-at-Large to fill a vacancy and re-elected February 1945; Chairman Committee 613, which reported "Recommended Practice for the Design of Concrete Mixes,"

now become a Standard of the Institute. Has written and participated in the writing of many technical contributions to ACI work.

Director-at-Large (to succeed himself)

HENRY L. KENNEDY, Manager Cement Section, Dewey & Almy Chemical Co., Cambridge, Mass. ACI Member since 1934; elected to Board of Direction 1944 and re-elected 1945, and at once appointed to fill a vacancy as Director-at-Large caused by the election of Stanton Walker to a vice-presidency.

Regional Director, First District (to succeed himself)

PAUL W. NORTON, Consulting Engineer, Boston, ACI Member since 1931; appointed First-District Director to fill vacancy caused by Mr. Kennedy's appointment as Director-at-Large.

Regional Director, Second District

ROY R. ZIPPRODT, Research and Consulting Engineer, Committee on Reinforced Concrete Research, American Iron & Steel Institute, ACI Member since 1920 and Secretary ACI Committee 318, Standard Building Code.

Regional Director, Third District (to succeed himself)

ALEXANDER FOSTER, JR., Vice-President, Warner Co., Philadelphia; ACI Member since 1912; elected to the Board as Director Third District, 1945.

Regional Director, Fourth District (to succeed himself)

FRANK H. JACKSON, Senior Engineer of Tests, Public Roads Administration, Washington, D. C.; ACI Member since 1924; a member Advisory Committee as Chairman Department 900, Joint Efforts, since 1929; member Program Committee 1938-40; member Publications Committee since 1941; member Board of Direction as Director Fourth District, 1937-38 and elected to the same position, 1945.

Regional Director, Fifth District (to succeed himself)

CHARLES S. WHITNEY, Consulting Structural Engineer, Milwaukee, Wis., ACI Member since 1920; Wason Medalist for Most Meritorious Paper; elected to the Board as Director Fifth District 1945.

Regional Director, Sixth District (to succeed himself)

HERBERT J. GILKEY, Head Department Theoretical and Applied Mechanics, Iowa State College, Ames, Ia.; ACI Member since 1924; Wason Medalist for Most Meritorious Paper; Member Standards Committee and chairman since March 1944; member Publications Committee since 1939; member Board of Direction, as Director Sixth District, 1937-38 and elected to the same position 1945.

Nominating Committee

The Nominating Committee also presented 20 candidates from among whom five are to be chosen to serve with the three latest past-presidents as the 1946 Nominating Committee—the candidate receiving the most votes on the letter ballot to be Chairman. The 20 candidates:

ARTHUR J. BOASE
S. J. CHAMBERLIN
MILES N. CLAIR
P. J. FREEMAN
FRED HUBBARD
J. W. KELLY
WILLIAM F. KELLERMAN
GEORGE A. MANEY
H. S. MEISSNER
JOHN R. NICHOLS
F. V. REAGEL
FRANK R. RICHART
J. R. SHANK
B. W. STEELE
E. O. SWEETSER
LOUIS H. TUTHILL
I. L. TYLER
C. A. WILLSON
BENJAMIN WILK
ROY N. YOUNG

Who's Who in this ACI Journal

Clifford A. Betts

is engineer U. S. Forest Service, and not a member of this Institute, but he writes interestingly of forest service concrete work (p. 1). He is a Ph. B. of the Sheffield Scientific School of Yale University and was a scholarship student in hydraulics at the University of Wisconsin.

As a student he did surveying and city map work for South Norwalk, Conn. where he was later inspector on sewer construction, then draftsman on waterworks and sewage disposal plans with James H. Fuertes, New York; in charge of building construction in South Norwalk; he did research work in the hydraulic laboratory, University of Wisconsin, 1914; was engineer for the City of Norwalk, Conn. in 1915; assistant city engineer in Norwalk in 1916; assistant engineer, Bridgeport, Conn. 1917; did design work and drafting for the Commissioner of Public Docks, Portland, Oregon; then was chief engineer for the Cummings-Moberly Lumber Co. cooperating with the U. S. Government in aircraft spruce production, 1918; with United States Forest Service, Denver, Denver Municipal Water works, 1919 to '23. He was office engineer, Moffatt Tunnel Commission 1923 to '28; engineer, Bureau of Reclamation on Owyhee Dam, Oregon and on 3½ and 4½ mile long tunnels 1928 to '34. On the technical staff of the Mississippi Valley Commission and National Resources Commission for comprehensive planning for utilization of water resources, 1934; engineer, U. S. Forest Service, Washington, D. C. 1935 to date. He has written papers for the engineering press and won the Thomas Fitch Roland prize 1932, American Society Civil Engineers.

Ralph W. Kluge and Edward C. Tuma

collaborated in the paper on lapped-bar splices (p. 13).

Mr. Kluge, an ACI Member since 1938 is not only not new to these pages, but is a Wason medalist by reason of his work with Wilbur M. Wilson in "Tests of Rigid Frame Bridges." He left the University of Illinois where he was Special Assistant Professor of Theoretical and Applied Mechanics, in 1941 to go to the Bureau of Standards, as structural engineer in the Masonry Construction Section.

Mr. Tuma was graduated from the Nebraska State Teachers College with the degree of B. S. in 1930. After graduation, he was employed for short periods with the Rocky Mountain Drilling Co. and the Kansas City Bridge Construction Co.; taught mathematics at the Big Horn County High School in Lovel, and later at the Hot Springs County High School

at Thermopolis, Gotham, Wyoming. In January 1943, he was appointed Jr. Materials Engineer in the Masonry Construction Section of the National Bureau of Standards.

Culbertson W. Ross

(see p. 37) received his B. S. degree from Alma College in 1926 and in 1931 his M.S. from the University of Michigan. He has been at the National Bureau of Standards since 1927, where he was first employed in the Cement Section, and since 1930 in the Masonry Construction Section where he has been engaged in the study of the deflections of structures with temperature changes; in connection with the Arlington Memorial Bridge, and the Reinforced Concrete Arches of the Navy Model Testing Basin at Carderock, Md.

Charles E. Wuerpel

who reports extensive field use of cement containing Vinsol resin (p. 49) is not new to these pages. He is Principal Engineer, Central Concrete Laboratory, North Atlantic Division Corps of Engineers U. S. Army and has been an ACI Member since 1937.

New Members

The Board of Direction approved 78 applications for Membership (66 Individual, 6 Corporation, 3 Junior, 3 Student) received in May, June and July as follows:

J. Francisco Aguilar V., 1845 Franklin St., San Francisco 9, Calif.

Felix Garcia Alonso, 22 Nro. 154 Vedado, Havana, Cuba

Edwin C. Anderson, 986 Dexter Horton Bldg., Seattle 4, Wash.

Jose Pernz Benitoa, Ave 5a No. 8 Rpto, Miramar, Havana, Cuba

Emanuel Ben-Zvi, Kiryath Shmuel, House Gruber, Jerusalem (Palestine)

Ross D. Billings, United States Bureau of Reclamation, Anderson Dam, Idaho

Donald Blair, 173 Daly Ave., Ottawa, Ontario, Canada

H. C. Boyd, c/o Virginia Steel Co., P. O. Box 595, Richmond, Va.

Servando Pita Camacho, Patrocinio 59 Vibora, Havana, Cuba

Nicanor del Campo, 14 y 19 Rpto, Almendares, Marianao, Havana, Cuba

James T. Caprio, 1221 N. Fremont Ave., Tucson, Ariz.

Fco. Martin y Ruiz del Castillo, Bentre 6 y F. Rpti Beniteg, Marianas, Havana, Cuba

Ceco Steel Products Corp., 5701 West 26th St., Cicero 50, Ill. (H. D. Jolley)

Cement & Concrete Assn., 52 Grosvenor Gardens, London, S. W. 1, England

Thomas F. Chace, 268 Market Street, San Francisco 11, Calif.

R. S. Chew, 936 Mills Bldg., San Francisco, Calif.

Walter L. Couse, 12740 Lyndon Ave., Detroit 27, Mich.

Walter A. Crossman, 216 South Lee, Altus, Okla.

G. L. Cummings, c/o C. D. Howe Co. Ltd., Port Arthur, Ontario, Canada

Stuart B. Dickens, 13 Kingsway Road, Leicester, England

Gamil Boulos Fanous, 4 Harass St., Garden City, Cairo, Egypt

Martin E. Flaherty, 9205 Whitney Ave., Elmhurst, N. Y.

W. H. G. Flay, 386 Sunnyside Ave., Ottawa, Canada

Herman Frauenfelder, P. O. Box 606, 166 Chapel St., New Haven 3, Conn.

G. W. Froggatt, Borough Engineers Office, Town Hall St., Blackpool, England

R. H. Gagle, Montana Highway Dept., Helena, Mont.

Alex Graham Garvock, c/o A. I. Garvock Ltd., Regent Theatre Bldg., Ottawa, Ontario

Ray C. Giddings, 4669 East 49th St., Los Angeles, Calif.

Great Lakes Concrete Pipe Co., Inc., 9 Austin St., Buffalo 7, N. Y. (M. C. Kelly)

Robert H. Griffin, 5909 Contra Costa Rd., Oakland 11, Calif.

Joseph R. Guptill, 5936 McAndrew Drive, Oakland 11, Calif.

Ronald W. Hadley, Ceco Steel Products Corp., Foot of New York Ave., Jersey City, N. J.

Lester T. Hagadorn, P. O. Box 518, Salina, Kansas

Leo Hirsh, c/o Pacific Concrete Products Co., 11330 Tuxford, Roscoe, Calif.

Lyman G. Horton, 191 S. El Molino Ave., Pasadena 5, Calif.

Joyce & McGregor Ltd., P. O. Box 1424, Cape Town, South Africa

Jose Lecuona, Aguiar No. 361 Dpto. 202, Havana Cuba

William Lerch, Portland Cement Assn., 33 W. Grand Ave., Chicago 10, Ill.

Walter Lohrey, 788 Central Ave., Scarsdale, N. Y.

W. T. MacDonald, 16 Gould St., Toronto, Ontario

Thomas C. McPoyle, Birdsboro, Pa.

L. E. McQuillin, Dewey Portland Cement Co., Dewey, Okla.

H. L. Mahaffy, 355 St. James St., Montreal, Canada

S. I. Mahbub, K. B., c/o The Registrar Irrigation Secretariat, Lahore, India

Cesar E. Maso, San Lazaro 682, Havana, Cuba

H. B. Miller, c/o Christiani & Nielsen, P. O. Box 2827, Cape Town, S. Africa

Sylvester Morabito, Hq. Det. 2233 Engr. Const. Ser. Section APO 244, c/o P.M. San Francisco, Calif.

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Charles I. Orr, 222 Noble Ave., Akron 2, Ohio

Pan American Commerce, Inc., 205 W. Wacker Dr., Chicago, 6, Ill. (Thomas Mabry)

Mario Romanach Paniagua, 19 y 6 Apt F Vedado, Havana, Cuba

Adrian del Paso Jr., Iturrigaray 160, Mexico, D. F.

Frederick Payne, Portland House, Tothill St., London, S. W. 1, England

Frederick James Pearson, 12 Torkington Road, Gatley, Cheshire, England

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William M. Robb, Abbey House, Castlegate, Grantham Lincs, England

Simeon Ross, 6544 Saunders Street, Forest Hills, L. I., N. Y.

Tilton E. Shelburne, Va. Dept. of Highways, Richmond 19, Va.

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Manuel Angel Gonazlez del Valle, Oficios No. 104, Apt. 304, Havana, Cuba

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J. W. Wynn, c/o Engstrom & Wynn, 1117 Chapline St., Wheeling, W. Va.

Chia Shiang Yen, 1105 W. Clark St., Urbana, Ill.

New ACI Charter

ACI has been re-incorporated with a new statement of objects. The Institute's original charter, granted to its predecessor organization, the National Association of Cement Users, by articles of incorporation, recorded in the District of Columbia, Dec. 20, 1906, (first convention 1905) was amended for change of name to American Concrete Institute July 2, 1913. Steps to a new charter, better to set forth the developed purposes of the organization, were completed with re-incorporation Aug. 8, 1945.

A certified copy of an excerpt from minutes, Board of Direction, Feb. 16, 1945 is the basis of and embodies the new articles. From the minutes:

11) WHEREAS, a referendum ballot by Members of the American Concrete Institute on charter revision showed 576 for and 2 votes against the proposition with 107 members not voting on this proposition,

AND WHEREAS, the proposed revision of the Articles of Incorporation of the National Association of Cement Users, under the Incorporation Laws of the District of Columbia, recorded December 20, 1906 and as amended with change of name to American Concrete Institute July 2, 1913, was further revised by the affirmative vote of more than two-thirds of the members of the Board of Direction, February 29, 1944,

THEREFORE, Past Presidents Roy W. Crum, P. H. Bates and Ben Morcell, all of whom are citizens of the United States and Mr. Bates and Vice Admiral Morcell, who are residents of the District of Columbia, are now authorized to act for the American Concrete Institute in filing the approved Certificate of Reincorporation as follows:

Know all men by these presents:

That we the undersigned, of whom all are citizens of the United States and a majority are citizens of the District of Columbia, have been duly authorized by affirmative vote of more than two thirds of the members of the Board of Direction of the American Concrete Institute, a corporation existing under the laws of the District of Columbia, to file this Certificate of Re-incorporation, under Section 29-604 of Chapter VI, of Title 29 of the Code of the District of Columbia (1940).

1. **The name of the corporation shall continue to be the American Concrete Institute.**
2. **The existence of said corporation shall be perpetual.**
3. **The objects of the said corporation shall be to organize the efforts of its members for a non-profit public service in gathering, correlating and dissemi-**

nating information for the improvement of the design, construction, manufacture, use and maintenance of concrete products and structures. To this field of engineering education its resources shall forever be devoted.

4. **The management of the corporation for the ensuing year shall be vested in a Board of Direction, consisting of eighteen members.**

I, the undersigned, do hereby certify that I am the duly appointed Secretary-Treasurer of the American Concrete Institute, and that the foregoing is a true and correct excerpt from the minutes of a meeting of the Board of Direction of the said corporation held on the 16th of February 1945, at which meeting a quorum was present and that more than two-thirds of the eighteen members of the Board voted in favor of the resolution.

IN WITNESS WHEREOF, I have hereunto set my hand and have caused the seal of the corporation to be affixed this 17th day of July, 1945 AD.

HARVEY WHIPPLE, Secretary-Treasurer

Signed by Past Presidents—Ben Moreell, P. H. Bates and R. W. Crum, July 26, 1945, the new Articles of Incorporation were recorded, District of Columbia, Aug. 8, 1945.

Honor Roll

February 1 to August 31, 1945

Rene Pulido y Morales, in Havana, Cuba heads the list with 16 new Members proposed since Feb. 1.

Rene Pulido y Morales	**16**
Roy Zipprodt	**5**
Harry B. Dickens	**4**
H. F. Gonnerman	**4**
A. Amirikian	**3**
J. H. Spilkin	**3**
Charles S. Whitney	**3**
Ernst Gruenwald	**2½**
Charles E. Wuerpel	**2½**
C. Blaschitz	**2**
Francis MacLeay	**2**
D. E. Parsons	**2**
J. M. Wells	**2**
J. W. Kelly	1½
W. Fisher Cassie	1
A. R. Collins	1
H. W. Cormack	1
R. F. Dierking	1
G. H. Hodgson	1
V. P. Jensen	1
L. I. Johnstone	1
William G. McFarland	1
Denis Matthews	1
Calvin C. Oleson	1
A. F. Penny	1
Kenneth Powers	1
John A. Ruhling	1
C. H. Scholer	1
Byram Steel	1
G. W. Stokes	1
Wm. Summers, Jr.	1
M. A. Timlin	1
J. W. Tinkler	1
Maxwell Upson	1
Stanton Walker	1

The following credits are, in each instance, "50-50" with another Member:

Birger Arneberg	R. E. McLaughlin
E. E. Bauer	Adolph Meyer
E. W. Bauman	M. D. Olver
R. R. Coghlan	F. E. Richart
R. B. Crepps	Kanwar Sain
Harmer E. Davis	J. L. Savage
W. C. Hanna	Oskar Schreier
Carl W. Hunt	A. L. Strong
Paul A. Jones	A. R. Waters
H. J. McGillivray	K. B. Woods

Lloyd P. Lumpkin

a member of the Institute from 1937; died at his home in Little Rock, Ark., Jan. 11, 1945—word of the death having only recently came to the Institute. He was a native of Dixon, Missouri; was graduated from the University of Missouri and became engineer of materials and tests for the Arkansas State Highway Department; later technical engineer for the Marquette Cement Mfg., Co. Chicago; also associated with Ben M.

Hogan Co. general contractors, Little Rock.

Frederick H. Richardson

who had been a member of the Institute since 1932 died November, 1944 according to word reaching the Institute only recently. Mr. Richardson was born in Detroit in 1882; he was educated in Seattle Public Schools and the University of Washington (Sigma Nu). He was in general engineering practice from 1906 to 1916, was captain of Engineers AEF in France 1918 to '19; senior examiner, U. S. Shipping Board, Emergency Fleet Corp.; Assistant Engineer in Washington State Highway Department; District Engineer, Portland Cement Association; Consulting Engineer Portland Cement Association of Utah some time prior to his death.

Fred A. Davis

who had been a member of the Institute since 1929 and Materials Engineer and head of the Department of Tests of the State Road Commission of West Virginia, died December 17, 1944, in a Morgantown hospital after a long illness. Word of his death only recently came to the Institute. Mr. Davis had joined the road commission as assistant engineer in 1921 and after serving several terms, as City Engineer of Morgantown which was his birth place. He was graduated C. E., from West Virginia University in 1898; started his engineering work in the employ of the Baltimore & Ohio Railroad; he was consulting engineer during the building of the "short line" from Clarksburg to New Martinsville. At one time he was connected with Clark and Krebs, Charleston engineering firm, developing coal lands; was later with the Pittsburgh engineering concern of Heyl & Patterson. He was also a member of the American Society for Testing Materials.

Lith-I-Bar Associates

manufacturers of concrete joists of the Lith-I-Bar type have organized to develop and promote their concrete joist construction for floors and roofs. Members of the organization include: The Formigli Corp. Philadelphia, Pa., The Dextone Co., New Haven 3, Conn., Gravel Products Corp., Buffalo, N. Y., Otto Buehner & Co., Salt Lake City 5, Utah, Lith-I-Bar Co. of Calif., Los Angeles, Calif., Economy Cast Stone Co., Richmond, Va., Arnold Stone Co., Greensboro, N. C., Cambridge Cement Stone Co., Boston, Mass. Office headquarters are in New Haven, 3, Conn. P. O. Box 606 with Herman Frauenfelder, Secretary.

The 42nd Annual

ACI CONVENTION

New Yorker Hotel

New York City

February 18 to 21

1946

Have You a Volume X ACI Proceedings to Spare?

The civil engineering department of a university needs Vol. X, 1914, Proceedings of the American Concrete Institute to complete its set. Somebody may have an extra copy, and if so, the Institute would be glad to know it and if it's in good condition, to pay the full membership price of the volume—$5.00.

This Journal is very late due to acute man power shortage at our printers. We are sorry. The releases from the armed services and the war plants apparently have not included many printers.

ACI publications in large current demand

ACI Standard—1945

148 pages, 6x9 reprinting ACI current standards: Building Regulations for Reinforced Concrete (ACI 318-41); three recommended practices: Use of Metal Supports for Reinforcement (ACI-319-42); Measuring, Mixing and Placing Concrete (ACI 614-42), Design of Concrete Mixes (ACI 613-44), and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—all between two covers, $1.50 per copy—to ACI Members, $1.00.

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer" and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944. $1.25 per copy; 75 cents to Members.

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940; 140 pages. Price $1.50—to ACI members $1.00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

Concrete Primer (Feb. 1928)

Prepared for ACI by F. R. McMillan, it had five separate printings by the Institute alone (totalling nearly 70,000 copies). By special arrangement it has been translated and published abroad in many different languages. It is still going strong. In the foreword the author said "This primer is an attempt to develop in simple terms the principles governing concrete mixtures and to show how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete." 46 pages, 25 cents (cheaper in quantity).

For further information about ACI Membership and Publications (including pamphlets presenting Synopsis of recent ACI papers and reports) address:

AMERICAN CONCRETE INSTITUTE 742 New Center Building Detroit 2, Michigan

Sources of Equipment, Materials, and Services

A reference list of advertisers who participated in the Fourth Annual Technical Progress Issue of the ACI JOURNAL—the pages indicated will be found in the February 1945 issue and (when it is completed) in V. 41, ACI Proceedings. **Watch for the 5th Annual Technical Progress Section in the February 1946 JOURNAL.**

Concrete Products Plant Equipment

page

Besser Manufacturing Co., 800 45th St., Alpena, Mich. 428
—Concrete products plant equipment, production

Stearns Manufacturing Co., Inc., Adrian, Mich. 387
—Vibration and tamp type block machines, mixers and skip loaders

Construction Equipment

Baily Vibrator Co., 1526 Wood St., Philadelphia 2, Pa. 407
—Concrete vibrators

Blaw-Knox Division of Blaw-Knox Co., Farmers Bank Bldg., Pittsburgh, Pa. 394-5
—Truck mixer loading and bulk cement plants, road building equipment, buckets, batching plants, steel forms

Butler Bin Co., Waukesha, Wis. 421
—Central mix, ready-mix, bulk cement and batching plants, cement handling equipment

Chain Belt Co. of Milwaukee, Milwaukee, Wis. 418-9
—Mixers, pavers, pumps

Electric Tamper & Equipment Co., Ludington, Mich. 410-11
—Concrete vibrators

Flexible Road Joint Machine Co., Warren, Ohio 389
—Pavement joint and joint installers

Fuller Co., Catasauqua, Pa. 377
—Unloading and conveying pulverized materials

Heltzel Steel Form & Iron Co., Warren, Ohio 378-9
—Pavement expansion joint beams

Jaeger Machine Co., The, Columbus, Ohio 392-3
—Concrete paving equipment

C. S. Johnson Co., The, Champaign, Ill. 429
—Mixing and batching plants, buckets, elevators

Kelley Electric Machine Co., 287 Hinman Ave., Buffalo 17, N. Y. 422-3
—Floor finishing equipment

Koehring Co., Milwaukee, Wis. 380
—Tilting and non-tilting construction mixers

Mall Tool Co., 7703 So. Chicago Ave., Chicago 19, Ill. 427
—Concrete vibrators

Master Vibrator Co., Dayton 1, Ohio 430-1
—Concrete vibrators

Ransome Machinery Co., Dunellen, N. J. 433
—Mixers—paving, truck

Viber Co., 726 So. Flower St., Burbank, Calif. 381
—Concrete vibrators

Contractors, Engineers and Special Services

Kalman Floor Co., Inc., 110 E. 42nd St., New York 17, N. Y. 390-1
—Floor finishing methods

Prepakt Concrete Co., The, and Intrusion-Prepakt, Inc., Union Commerce Bldg., Cleveland 14, Ohio 397-400
—Pressure filled concrete

Raymond Concrete Pile Co., 140 Cedar St., New York 6, N. Y. 396
—Pile foundations

Roberts and Schaefer Co., 307 No. Michigan Ave., Chicago 1, Ill. 426
—Thin shell concrete roofs

Scientific Concrete Service Corp., McLachlen Bldg., Washington, D. C. 416
—Mix controls and records

Vacuum Concrete, Inc., 4210 Sansom St., Philadelphia 4, Pa. 409
—Suction control of water in the concrete

Materials and Accessories

Calcium Chloride Assn., The, 4145 Penobscot Bldg., Detroit 26, Mich. 420
—Calcium chloride

Concrete Masonry Products Co., 140 W. 65th St., Chicago, Ill. 385
—Non-shrink metallic aggregate

Dewey and Almy Chemical Co., Cambridge, Mass. 412-5
—Air-entraining and plasticising agents

Horn Co., A. C., Long Island City 1, N. Y. 417
—Waterproofing

Hunt Process Co., 7012 Stanford Ave., Los Angeles 1, Calif. 425
—Curing compound

Inland Steel Co., The, 38 So. Dearborn St., Chicago 3, Ill. 384
—Reinforcing bars

Lone Star Cement Corp., 342 Madison Ave., N. Y. 382-3
—Portland cements

Master Builders Co., The, Cleveland, Ohio, Toronto, Ont. 402-5
—Cement dispersing and air-entraining agents

Rail Steel Bar Association 386
—Reinforcement bars

Richmond Screw Anchor Co., Inc., 816 Liberty Ave., Brooklyn 8, N. Y. 408
—Form tying devices

Sika Chemical Corp., 37 Gregory Ave., Passaic, N. J. 434-5
—Waterproofings, plasticizer, and densifier

United States Rubber Co., Rockefeller Center, New York 20, N. Y. 401
—Form lining

Testing Equipment

American Machine & Metals, Inc., East Moline, Ill. 388
—Riehle hydraulic testing machines

ACI Standards—1945

(collected in one publication)

Standards of the American Concrete Institute adopted since the inauguration of the current procedures for their consideration and promulgation under the supervision of the Standards Committee are available in a single publication, as reprinted from the Journal of the American Concrete Institute, June 1945, pages 559-704. Each Standard is also available in a separate print. New editions of the collected ACI Standards will be issued as rapidly as is justified by the completion of technical committee work. Some of the present Standards have had some few editorial revisions. They include changes in the substance of the texts as approved by the ACI Conventions which adopted them and as subsequently ratified by letter ballot of the ACI Membership.

Not included are "proposed standards" presented in recent years, nor proposed or ratified Standards prior to 1937. Some of the latter will have thorough review and eventually come before the Institute for further consideration. The book contains

	Pages
Building Regulations for Reinforced Concrete (ACI 318-41)	**559-620**
Recommended Practice for the Use of Metal Supports for Reinforcement (ACI 319-42)	**621-624**
Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)	**625-650**
Recommended Practice for the Design of Concrete Mixes (ACI 613-44)	**651-672**
Specifications for Concrete Pavements and Bases (ACI 617-44)	**673-700**
Specification for Cast Stone (ACI 704-44)	**701-704**

(148 pages in covers $1.50—to ACI Members $1.00)

These Standards are also available in separate prints in covers at 50 cents each (cheaper in quantity and to ACI Members) except that ACI 319-42 and ACI 704-44, not in covers, are 25 cents each.

AMERICAN CONCRETE INSTITUTE
New Center Building, Detroit 2, Michigan, U. S. A.

[To facilitate selective distribution, **separate prints** of this title (42-5) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than March 1, 1946.]

Title 42-5 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

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Maintenance and Repair of Concrete Bridges on the Oregon Highway System*

By G. S. PAXSON†
Member American Concrete Institute

SYNOPSIS

Describes types of concrete disintegration and methods and materials used in Oregon, for repair and replacement and for protection against further deterioration.

When a bridge is completed and opened to traffic the construction phase is succeeded by the maintenance and operation phase. Our construction materials are all subject to deterioration. Some of these materials have longer service life than others, but eventually deterioration becomes evident in all of them. The term "permanent construction" is only relative. Concrete has always been regarded as one of the longest lived materials ordinarily used in bridge construction, but if its full service life is to be realized, maintenance is necessary.

Disintegration of structural concrete can roughly be divided into two classes: first, damage that is caused by outside agencies, as in traffic collision, settlement of foundations, overstress, or inadequate provision for temperature changes; second, disintegration due to some weakness of the concrete itself. This is usually evidenced by a disintegration of the surfaces or edges of the concrete members.

The procedures used in the repair of damage caused by outside agencies are as varied as the causes themselves. Chipped or broken members can be rebuilt or patched. The bonding of new to old concrete has been discussed frequently in the technical press. The method used in Oregon is given later in the description of the repair of disintegrated concrete surfaces. Foundation settlement can be corrected by underpinning and by raising the superstructure to its original position. The amount of

*Received by the Institute Sept. 4, 1945.
†Bridge Engineer, Oregon State Highway Department.

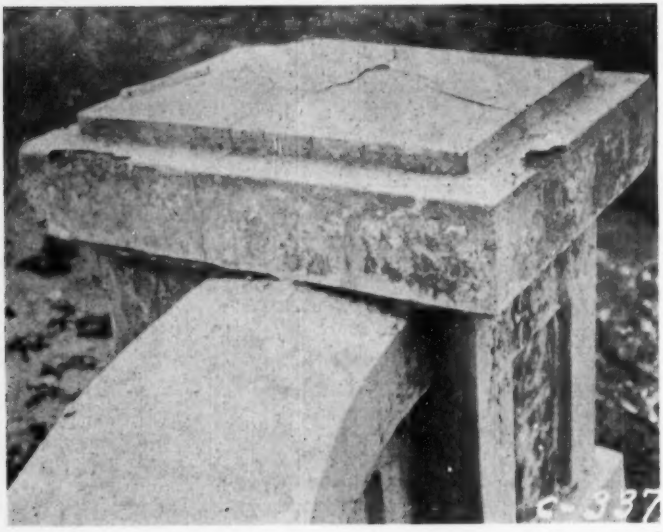


Fig. 1—Handrail posts at the Snake River Bridge.

Fig. 2—Culvert headwall at the Franklin Avenue Undercrossing in Bend.

distortion that concrete members will stand without serious injury is surprising in view of the general opinion that concrete is a stiff and brittle material. The cause of damage of this kind can be determined easily, and the repairs can be expected to effect a permanent cure. Deterioration caused by some weakness of the material itself is more difficult to treat. The cause of the trouble is obscure and remedial work is often only a temporary check to the deterioration.

The first alarming example of this second type of disintegration to be noticed on the Oregon highway system showed up on the Snake River Bridge at Ontario, Ore. This bridge consists of four 200-ft. steel truss spans with concrete deck, curbs and piers, and with a short concrete viaduct approach at each end. A steel handrail was used on the truss span and a concrete handrail on the approach spans. The bridge was built in 1924 and by 1928 wide cracks had opened up in the piers and in the larger members of the handrail. These cracks were peculiar in that little, if any, disintegration of the concrete accompanied the cracking. The cracks were relatively wide, an open width of as much as three-eighths inch was found, but the concrete along the edges of the crack appeared sound. Fig. 1 shows a massive handrail post with the wide cracks showing in the top. Fig. 2 shows a head wall of a culvert built ten years before the picture was taken. The cracking occurred in the first five years after the structure was built, and has shown little or no progress since that time. The fact that these cracks occur in members not subject to structural stress indicates a lack of any connection between stress and this type of cracking.

An attempt was made to seal the cracks in the Snake River Bridge piers with a heavy mixture of white lead and oil forced in under pressure. The continued opening of the cracks broke the seal, and the pier shafts and webs were encased with concrete in 1934. This new concrete was waterproofed in 1937 and no further trouble has occurred. The handrails were not encased, and the cracks have shown no increase in width for the past ten years. It seems that this type of cracking takes place during the first few years after the concrete is placed, and if not complicated by other types of disintegration, then ceases or at least progresses very slowly. There are a few examples of this type of cracking in the Willamette River Valley where only a few cycles of freezing and thawing occur each year. Fig. 3 shows a handrail post on the Willamette River Bridge at Albany in 1937. No repair work has to be done, and there has been no apparent change in the condition in the last eight years.

At the time these structures were built it was not common practice to make alkali determinations on the cement used. It so happened, however, that most of this kind of trouble developed in concrete made with cement manufactured during the first two years operation of one

Fig. 3—Handrail post at the Willamette River Bridge in Albany.

cement plant. Samples were obtained from the corners of a bin which probably represented the first cement stored. These samples showed an alkali content of 1.68 per cent, calculated as Na_2O. It is probable that this type of cracking can be attributed to the interaction of a high-alkali cement and a reactive aggregate, as no noticeable cracking of this type has been found when these same aggregates and cements running from 0.6 to 0.8 percent alkali were used. Structures built in the same area with the same aggregates and with cement from the same mill after a change in the source of raw materials, have shown no disintegration of this type.

When the cracking has been serious enough to necessitate repair, the surfaces have been chipped away and the members encased with concrete made with low-alkali cement and aggregates known to be nonreactive. The preparation of the surfaces and the placing of the concrete is carefully done to insure a good bond. Where an encasement exceeding four inches thick is placed, anchor dowels and mesh reinforcement are used.

The cause of this disintegration, which manifests itself by wide cracks, is quite clearly related to the alkali content of the cement and the reactiveness of the aggregates. The fact that it occurs in locations having mild climate and few cycles of freezing and thawing as well as in the colder climates indicates that temperature change is not the primary cause. After this cracking has occurred, moisture penetrates the concrete more easily and disintegration due to freezing and thawing is accelerated. This type of disintegration, while serious in the few struc-

tures that have been affected, is not of great concern in Oregon. The cause and means of prevention are known.

During the last 15 years another type of disintegration has developed that is much more serious than the type described above. The first indication of trouble usually shows up within four or five years after construction. A large number of fine cracks appear. These are closely spaced and parallel with the edges of the affected members. The cracks are filled with a grey deposit and are generally called "D-line" cracks. In so far as has been observed, this type of disintegration has been confined to the eastern section of the state and to the higher parts of the Cascade Mountains. It appears to be definitely a function of the number of cycles of freezing and thawing. In the western section of the state where a few cycles only of freezing and thawing occur annually, no trouble has been experienced.

This disintegration, so far, has been confined to handrails, curbs, wing wall tops, and other members above the roadway surface. Members which are protected by the roadway slab have not been affected. These observations point toward the conclusion that the tendency to disintegrate, due primarily to a weakness of the concrete itself, can be overcome, or at least retarded, by waterproofing the surfaces of members exposed to rain or snow.

One significant point brought out in the analysis of data from the frequent inspections of the bridges on the highway system is the correlation between the date of construction and the presence or absence of this "D" line disintegration. A tabulation of all concrete bridges on a 228-mile section of the Old Oregon Trail Highway (U. S. 30) between Umatilla and Ontario was made recently. There are 61 structures in this section. Of these 61 structures, 37 were built prior to 1930 and 24 were built from 1930 to 1937. Of the 37 older structures, only seven show evidences of "D-line" disintegration, and five of these were built with the high-alkali cement previously referred to. The disintegration on the other two structures occurs at the end of a wing wall and in a section of curb where laitance indicates improper placing. Of the 24 structures built from 1930 to 1937, 15 show "D" line disintegration in more or less degree, and four of the other nine structures were waterproofed within two years after their construction.

The construction of these 61 bridges covers a period of 20 years. In general aggregates from the same local sources were used. We are not ready to admit that the later methods of proportioning, placing, and curing are inferior to those used 20 years ago. The remaining ingredient is the cement. At about 1930, finer grinding of cement became general practice to meet the demand for higher early-strength concrete. The

strength was obtained, but it seems probable that it was at a sacrifice of durability.

By 1937 the extent of "D-line" disintegration in the highway structure was causing serious concern. The determination of the basic cause of the trouble is too complex a problem for a single state highway organization, and but little progress along this line has been made. A maintenance procedure has been developed, however, which has been quite successful in repairing damage that has already occurred and in preventing further progress of the disintegration. This procedure consists in the removal and replacement of the disintegrated concrete and waterproofing of the surfaces with a linseed oil and white lead paint coat. This procedure follows, in general, the suggestions made by F. R. McMillan, Director of Research of the Portland Cement Association.

The following is an outline of the procedure used by the Oregon State Highway Department in patching disintegrated concrete and waterproofing concrete surfaces for the prevention of disintegration.

When disintegration of the commonly called "D-line" type has taken place, it usually affects the edges and corners of curbs, handrails, wing walls, and other exposed members. To prevent further progress it is necessary to remove completely all disintegrated material, great care being taken to reach sound concrete beyond the extent of the disintegrated area. This can be accomplished by the use of hammers and chisels on small areas or by paving breakers or chipping hammers on larger areas. If the disintegration has reached an advanced stage, the complete removal and replacement of the affected portion of the structure may be necessary. The importance of the removal of all traces of disintegrated material cannot be overemphasized. Experience has shown that often the workman will remove all visible affected material, then place a patch on what was, in his judgment, sound concrete, only to discover later that the material adjacent to the patch continues to disintegrate.

After the removal of the disintegrated concrete a patch is applied, the success of which depends upon securing a bond to the parent concrete, overcoming the tendency of the patch to shrink after placement, and proper curing. All places to be patched should be chipped out to secure not less than three-fourths-inch thickness for the patch. The edges of the patch should be square and not feathered. All surfaces should be clean and rough so as to secure a good bond and should be saturated thoroughly by several applications of water. The preshrinkage of mortar is required for all patches. This is done by mixing the mortar well ahead of use and letting it stand. The time required for preshrinkage of mortar varies with the different brands of cement and conditions of temperature and humidity, and is best determined by experiment on the

Fig 4—Sidewalk on Burnt River Bridge before repairing.

Fig 5—Sidewalk on Burnt River Bridge after repairing.

job. In general the mortar thus preshrunk should be susceptible to use without the addition of water before reworking it for application. After the patch has been applied, proper care in curing must be taken by keeping the patch covered with wet burlap for six to ten hours, after which it can be covered with damp sand or burlap until the concrete has thoroughly hardened.

New concrete patches should be allowed at least two weeks to dry out before applying the waterproofing treatment. New concrete should be

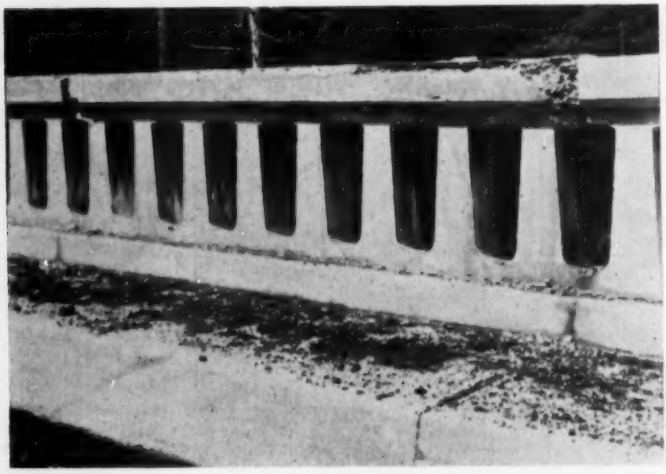
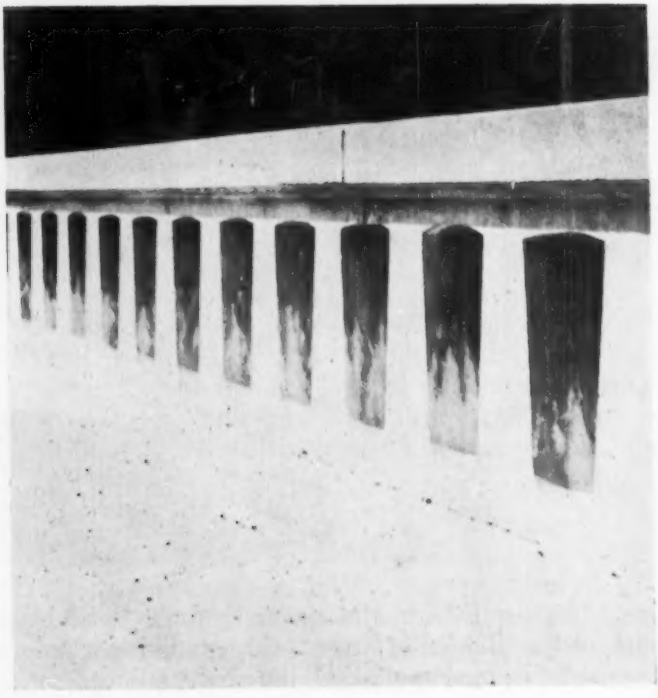


Fig. 6—Handrail on Santiam River Bridge before repairing.

Fig. 7—Handrail on Santiam River Bridge after repairing.



given a neutralizing wash before the application of the linseed oil used in the waterproofing treatment. A solution consisting of three pounds of zinc sulphate crystals to a gallon of water is brushed over the surface to be treated and allowed to dry for 48 hours. When thoroughly dry, all crystals on the surface are removed by wire brushing. This treatment is not necessary on old concrete.

Before the waterproofing treatment is applied, it is necessary that the concrete surface be clean and dry. Dust and loose material can be removed with a wire brush. Road oil or grease can be removed by scrubbing with gasoline or a solvent. Efflorescence can be removed by scrubbing with a ten per cent solution of hydrochloric acid. When water is used in cleaning, ample time must be allowed to permit thorough drying of the concrete surfaces before applying the waterproofing.

After the surface has been properly prepared and is clean and dry, two coats of hot linseed oil are to be applied. The first coat consists of a mixture of 50 per cent raw linseed oil and 50 per cent turpentine heated to 175 F. and applied with an ordinary paint brush. Better results will be obtained if the air temperature is above 65 F. The first coat is allowed to set 24 hours before applying the second coat. After the first coat of the linseed oil-turpentine mixture has set, spots will usually be noticed where the concrete is more porous than the remainder of the surface treated. These spots should be spot treated with the hot mixture and allowed to set before the second coat of linseed oil is applied.

The second coat consists of undiluted raw linseed oil heated to 175 F. and applied in the same manner as the first coat. When this coat is thoroughly dry, the surface is ready for a paint coat.

The entire surface treated with oil is given two coats of white lead and oil paint, tinted to the desired shade. A concrete color can be obtained by the addition of lampblack and raw sienna ground in oil. The white paint used in Oregon has the following formula:

Paint Composition	Per cent
Pigment not less than	70
Vehicle not more than	30
Pigment Composition	
White lead carbonate	40.0 to 45.0
Titanium barium pigment	35.0 to 40.0
Zinc Oxide	15.0 to 20.0
Tinting pigments, if required	0.0 to 5.0

The first coat is thinned by the addition of two quarts of turpentine and two quarts of boiled linseed oil to the gallon oa paint. The second coat is thinned with about one quart of boiled linseed oil to the gallon of paint so as not to give a heavy pigment coat that will be susceptible to scaling, but which is still heavy enough to brush out uniformly and evenly.

This waterproofing treatment is not a cure for the basic weakness of the concrete which makes it susceptible to disintegration. If properly applied and maintained, it promises to postpone serious disintegration for many years.

Fig. 4 shows a sidewalk where the edges have developed "D-line" disintegration and the concrete has crumbled away. Fig. 5 shows the same sidewalk after patching and waterproofing. Fig. 6 shows a handrail with "D-line" disintegration well advanced. Fig. 7 shows the same handrail after the affected concrete has been chipped out and the member built up with new concrete, but before the waterproofing was applied. These figures show that concrete members which have been seriously affected can be repaired and made serviceable and presentable.

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Should Portland Cement Be Dispersed?*

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Member American Concrete Institute

SYNOPSIS

A development of definitions of *wetting* and *dispersion* is followed by a discussion of dispersion of portland cement.

From elementary principles it appears that a wetting agent is unnecessary, for portland cement is highly hydrophilic.

The dispersed state of portland cement in water is defined as that state in which interparticle attraction in a fresh paste is absent or so weak that it has no appreciable effect on the physical properties of the fresh paste. Experiments and reasoning from general principles indicate that dispersion would be undesirable because it would increase the rate and amount of sedimentation and promote particle-size segregation in cement paste; it would destroy the plasticity of the pastes and give them the properties of a fluid, a probably undesirable change; it would have no beneficial effect on rate of hydration during the early stages through an increase in exposed surface area because the whole surface is normally exposed to water even when the particles are flocculated.

A reduction in interparticle attraction short of actual dispersion should reduce the water required for a given slump, but it would not improve workability except in unusually rich mixes. It would increase bleeding.

Air entrainment requires an increase in paste content and reduction of water content to maintain a given slump. It reduces strength but improves frost resistance. It improves workability and reduces bleeding.

Air entrainment together with some reduction in interparticle attraction affects paste content and water requirement in the same way as air entrainment alone, but the increase in paste content is smaller and the reduction in water content is greater than when there is no reduction in interparticle attraction. Air entrainment offsets the undesirable effects of reducing interparticle attraction on plasticity and reduces bleeding.

Some agents do not affect the chemical processes of hardening; their effects on strength can be predicted from the voids-cement ratio. Others tend to retard hydration unless they contain an accelerating agent. Such agents have different effects with different cements.

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INTRODUCTION

This discussion was prepared in response to many requests for information on various admixtures sold as dispersing agents. The questions are usually one of two types: (1) What is the effect of Admixture X on concrete? and (2) What is back of the idea of dispersion of cement? This discussion will deal primarily with the question of what is back of the idea of dispersing cement—not with the actual merits of preparations that have been sold as dispersing agents.

Some materials sold as dispersing agents may have merit because of effects not related to dispersion. However, they cannot be endorsed on the basis of some of the claims made for them. In explaining results obtained or hoped for the assumptions have been made that a normal cement paste is composed of individual flocs (clusters) of cement grains such as may be seen when a small amount of cement is suspended in a large volume of water; also that water for hydration does not penetrate the flocs and hence that the cement cannot hydrate as well as it might; that the water that is "trapped" in the flocs does not contribute to workability. It is then further assumed that the flocs of cement grains can be caused to disintegrate, that is, that the grains in a floc can be dispersed, by adding a suitable agent which causes the grains to acquire electrostatic charges and thus to become mutually repellent; and that when the cement grains are thus dispersed they hydrate more rapidly and to a greater extent than when flocculated. Moreover, it is assumed that when the particles are brought to a state of mutual repellency, the amount of bleeding or "settlement-shrinkage" is reduced. Dispersion is said also to improve resistance to frost action.

Most of these assumptions seem plausible, but they are not compatible with the results of recent researches on the nature of cement paste and on the whole are believed to be untenable on either a theoretical or empirical basis. The basis for this conclusion will be given briefly in this discussion.

The question to be discussed can easily become confused by the fact that some preparations used as dispersing agents are actually mixtures containing not only a dispersing agent but also an accelerator such as calcium chloride, and by the fact that most, if not all, such materials cause more or less air-entrainment. Yet *all* the effects of such mixtures have been attributed, by some, to the supposed dispersion of the cement particles. It is therefore necessary to remember that what may be said in the following discussion against the idea of dispersion does not necessarily impugn any particular preparation unless the predominant effect of the preparation is that of dispersing the cement.

Owing to the manner in which various types of materials of this class have been introduced to the concrete industry, there is some confusion

with respect to the terms "wetting agent," "dispersing agent," and "air-entraining agent." Therefore, a secondary aim of this discussion is to clear away some of the confusion with respect to terminology. It is especially important to grasp the fact that certain difficulties surround the use of the term "dispersion" as applied to portland cement. To bring out the nature of these difficulties, it is necessary to present a brief discussion of fundamentals. This discussion leads to a definition of dispersion of portland cement paste on which the final part of the paper is based. The reader is asked to avoid applying the final conclusions concerning dispersion as defined here, to dispersion defined in some other way.

The discussion of fundamentals will be found rather sketchy. For those seeking fuller information a guide to the literature is provided.

THE PHENOMENON OF WETTING*

When a drop of liquid is placed on a clean surface of a solid, it may either spread on the surface or remain as a more or less flattened drop, showing a definite angle of contact with the surface. When the liquid spreads spontaneously, wetting is said to be complete (contact angle = zero) and, if the liquid is water, the solid is called hydrophilic. If water does not spread but establishes a finite contact angle, wetting is incomplete or partial and the solid is regarded as hydrophobic.[15] Partial wetting, that is, the formation of a contact angle, denotes that the force of adhesion between the solid and the liquid (which tends to cause the liquid to spread over the surface) is less than the surface tension of the liquid, which tends to cause the liquid to gather itself into a drop. Conversely, wetting is always complete when the force of adhesion exceeds the surface tension of the liquid.

A wetting agent is usually a material composed of elongated organic molecules, the molecules having an affinity for the solid at one end and affinity for the liquid at the other. Such an agent forms a layer of molecules on the solid, the molecules being so oriented as to present a wettable surface to the liquid.

When the force of adhesion between a powdered solid and a liquid is less than the surface tension of the liquid, the powder and the liquid are difficult to mix unless a suitable wetting agent is used. Or if the adhesion tension exceeds the surface tension only slightly, a wetting agent will be noticeably helpful. On the other hand, if the solid and liquid show a strong mutual attraction, the liquid will spread over the solid surface without outside aid. For any given case the surface tension and adhesion tension may be measured and the need for a wetting agent

*See Ref. 1 at end of paper.

judged from the results. It is simpler, however, merely to observe directly whether wetting is complete or not. For example, if the specific gravity of the solid exceeds that of the liquid, small particles of the solid, when scattered on the surface of the liquid, will sink readily if the contact angle is very small or zero, or they will remain on the surface if the contact angle is sufficiently large, as does an oiled needle on water. In the case of portland cement and water it is still simpler merely to observe the absorption when water is placed in a crater of dry cement; capillary absorption is readily apparent unless the cement contains a "water-repellent" substance.

Any such criteria applied to portland cement would show that the wetting of portland cement by water could hardly be improved by a wetting agent, unless the cement has acquired a "water-repellent" coating. The degree of solubility of the constituents of cement and, in fact, the rapid formation of hydrates that occurs immediately on contact with water show that portland cement has a strong affinity for water. The attraction is so strong that each cement grain becomes completely surrounded by water even though in a dilute suspension the grains are clustered. This is shown by the fact that during sedimentation of a concentrated suspension, i.e., during bleeding, the *whole* surface area of the cement is effective in regulating the rate of flow of water.[2] So far as the writer knows, no one has seriously contended that cement needs a wetting agent.

When a powder is readily wetted, so that each grain becomes surrounded with water, the grains are necessarily separated, at least by a thin film of water. Because of this, the wetting of a powder by immersion in a liquid is sometimes referred to as dispersion. However, it is advantageous and in line with modern textbook terminology to draw a distinction between wetting and dispersion. As described above, the term *wetting* pertains to the spontaneous spreading of a liquid over a surface. It applies to any shape of surface, not to particles alone. Dispersion, on the other hand, pertains only to particles.* It is the opposite of flocculation, agglomeration, or coagulation. Wetted particles may be in either a flocculated or a dispersed state. Flocculated particles in a liquid may be held together by forces acting across separating films of liquid; hence, a flocculated state is not necessarily one requiring particle-to-particle contact.

These remarks give a general idea of what is meant by dispersion, flocculation, and wetting, but a definition of dispersion wholly adequate for the present discussion cannot be given without considering some additional details of the phenomenon.

*But not necessarily to *solid* particles only.

INTERPARTICLE FORCES

According to present-day theories, the interparticle force in an aggregation of particles in a fluid medium is made up principally of the following components:

(1) An ever-present force of attraction (van der Waal's forces) which causes adjacent particles to adhere; and

(2) An electrostatic force of repulsion that opposes the force of attraction. This repulsion is strongly dependent on the environment of the particles and is therefore subject to control.

The control of interparticle force may involve controlling the kind and concentration of electrolytes, or the use of certain kinds of organic molecules or colloids.

Because of the nature of the relationships between distance of separation and intensity of repulsion and intensity of attraction, two particles may have minimum potential energy when they are separated by a small but definite distance.[3, 4] Hence, even in the flocculated state, suspended particles may tend to remain slightly separated.*

DISPERSION

Spontaneous dispersion

Dispersion has been pictured as a spontaneous process whereby particles bearing like electrostatic charges "spring apart and stay apart" by distances easily observed with the microscope. However, even under the most favorable conditions, the forces of repulsion in a suspension are effective over only very short distances. Indeed, if the particles were separated only by the effective distance of repulsion, they would appear to be in contact under ordinary magnification. Nevertheless, if the particles are very small, they may be capable of dispersing themselves by their Brownian motion, far beyond the range of interparticle forces. Brownian motion is a haphazard movement, or vibration, caused by unbalanced impacts of the molecules of the surrounding medium. If a particle is small enough, the forces of the impacts received simultaneously from different directions do not balance and the particle thus acquires motion.

Particles having Brownian motion wander at random and tend to bounce away from each other when they collide. If the forces tending to keep the particles in motion exceed the force of attraction between the particles, then a state of dispersion will spontaneously be maintained. On the other hand, if the forces of attraction between the particles exceed the forces tending to keep the particles in motion, then a state of flocculation, or agglomeration, will persist. Note that interparticle

*For a thorough discussion of interparticle forces, see Ref. 5.

repulsion is not necessary for dispersion; but, of course, the more the interparticle attraction is cancelled by interparticle repulsion, the smaller the amount of kinetic energy required to keep the particles dispersed.

Brownian motion can occur to a significant degree only among very small particles—colloidal particles. Hence, portland cement cannot be caused to disperse spontaneously, for cement particles are predominantly microscopic, not colloidal.[6]

Mechanical dispersion

Even when spontaneous dispersion is not possible, dispersion can be effected mechanically. Stirring a suspension tends to separate the particles, particularly if the stirring is violent. The degree to which the particles will become separated on stirring depends largely on the intensity of the interparticle attraction. Of course, mutually repellent particles should be more easily dispersed than those which have some tendency to stick together.

* * *

The discussion immediately above was presented to show that some of the phenomena pictured in connection with the use of dispersing agents with portland cement actually can occur only among particles that are of truly colloidal dimensions. It must be added that whether or not spontaneous dispersion takes place is of little practical consequence. *The important question is whether it is necessary or desirable to cancel or reduce the forces of interparticle attraction that normally predominate over the forces of repulsion in a suspension of cement particles in water.*

INTERPARTICLE ATTRACTION AND PASTE PROPERTIES

Although the influence of interparticle attraction on the physical properties of pastes will be discussed more fully in the following section, two important effects may be mentioned at this point. The first is that the greater the interparticle attraction, the stiffer will a paste seem to be when it is stirred. In comparatively concentrated suspensions such as cement pastes, where interparticle attraction predominates over repulsion, the suspension behaves more like a solid than a fluid.[7a, 7b*] The other effect appears when suspensions are allowed to settle. If the forces of attraction predominate, the large and small particles settle together. If the forces of repulsion predominate or if the net force of attraction is very weak, particles that would remain in contact when quiescent become separated as they fall through the liquid during sedimentation.

The cause of this may be seen by considering two particles of different size adhering to one another at the beginning of their fall through the

*It behaves like a solid in that it is capable of withstanding small shearing forces. See page 129, "Effect of Flocculation on Plasticity."

liquid. The drag of the liquid on the small particle will be greater per unit mass than that on the large one; therefore, a force tending to separate them will develop. If this force, which depends upon the difference in the size of the particles, exceeds the force of attraction, the particles will separate, the larger particles falling more rapidly. When the attractive forces are very weak, and especially when the particles are mutually repellent, the sediment that is formed tends to be non-uniform in composition, the proportion of coarse particles increasing toward the bottom of the sediment. Moreover, the sediment formed is compact and difficult to redisperse, whereas a flocculated sediment is bulky, soft, and easily restored to its original state in the suspension. This feature of the behavior of the dispersed suspension is very significant with respect to the question of dispersing cement, as will be developed below.

DEFINITION OF DISPERSION APPLICABLE TO CEMENT PASTE

The foregoing discussions serve to show that a criterion for dispersion applicable to colloidal suspensions is not altogether applicable to temporary suspensions of non-colloidal particles; dispersion cannot mean exactly the same thing for both types of suspension. Yet circumstances seem to require using the same term for both cases; indeed, there is considerable justification for it. The dilemma is avoided by thinking in terms of interparticle attraction instead of the repulsion that is implied by the word "dispersion." A suitable definition of dispersion can be set up in terms of the influence that interparticle attraction has on certain properties of suspensions, particularly, the force-flow relationship. When interparticle attraction is absent or negligible, a suspension that is not too concentrated flows like a true fluid; but when interparticle attraction is not negligible, the suspension acquires the properties of a plastic solid to some degree. Also, in a *dilute* suspension of particles, segregation of sizes takes place during sedimentation if the interparticle attraction is absent or weak and it does not take place if interparticle attraction is strong. In view of such observations as these the following definition of dispersion is used in the discussion of cement paste that follows:

> *When interparticle attraction in a fresh cement paste is so weak that it has no appreciable effect on the behavior and physical properties of the paste, the particles in the paste may be said to be dispersed.*

By this definition we would call any suspension dispersed in which the interparticle attraction is zero or negative. This would not disagree with other definitions applied to colloidal suspensions. But we would also call a suspension dispersed if the interparticle force was positive, but too weak to have an appreciable effect on the physical properties of

the suspension. It should be noted that the definition does not rest on the presence or absence of particle-clusters; neither does it imply that dispersion is a spontaneous process.

This definition admits the possibility of various degrees of interparticle attraction among the particles in flocculated suspensions, that is, suspensions in which the attraction has an effect on flow-properties, etc. Consequently, we can discuss two questions, one pertaining to the desirability of producing dispersion, and one pertaining to the desirability of changing the intensity of interparticle attraction in a flocculated paste.

DISPERSION OF PORTLAND CEMENT

Flocculated state of normal portland cement paste

As implied at various points in the foregoing discussion, there is no question but that cement particles in a normal paste are flocculated. This may be seen by microscopic examination of cement-water mixtures sufficiently dilute to transmit light or by observation, at low magnification, of the texture of pastes through the wall of a glass container.

The process by which cement becomes flocculated may be visualized as follows: Under the action of a mechanical mixer, cement particles probably tend to be dispersed during the first few seconds of contact with the water. Chemical reactions begin immediately and continue at a relatively high rate for a period of not over five minutes—probably less than two minutes—during which time the electrolyte concentration in the mixing water increases rapidly. The electrolytes (apparently the hydroxyl ions) bring about flocculation of the cement particles.

During the same period a coating of hydrates forms on the cement grains. Once this coating has formed, and the electrolyte solution has reached full strength, the rate of reaction becomes very low and thus the cement remains comparatively dormant chemically. This state lasts for a considerable period, usually about an hour. The paste remains plastic during this period, and, if left undisturbed, undergoes sedimentation ("bleeding").

EFFECT OF FLOCCULATION ON AMOUNT OF SETTLEMENT

Except for certain features that need not be discussed here, the sedimentation of cement pastes has been shown to be essentially like that of other concentrated suspensions of flocculated mineral powders. Various experiments have been carried out that reveal the effect of flocculation on this process. Steinour[25] observed the sedimentation of emery particles both flocculated and not flocculated. Typical results are given in Table 1:

TABLE 1

Material	Diameter, microns	Fluid Content, %	Amount of Final Settlement, % of original height	
			Dispersed	Flocculated
Emery A	12.2	65	36	17
A	12.2	80	63	44
B	9.6	75	55	35

As shown in the last two columns of the table, flocculation *reduced* the amount of settlement at all concentrations.

The amount of settlement is influenced by the concentration of the flocculating agent. This is deduced from data such as the following obtained by the writer.[8]

TABLE 2—SEDIMENTATION OF PULVERIZED SILICA IN LIME-WATER
(60 ml of solution per 100g of silica)

Initial Concentration of $Ca(OH)_2$ in Mixing Water	Rate of Settlement cm/sec x 10^6	Final Settlement per Unit of Original Height, %
¼ saturated solution	58	10
½ saturated solution	53	5
saturated solution	53	1

Steinour[9] in tests on another silica-powder observed similar but less pronounced effects, the settlement being 12 and 19 per cent for saturated and 1/6-saturated solutions, respectively, with the initial fluid content at 60 per cent.

Results of some experiments with portland cement are given in Fig. 1. This shows the volumes of sediment formed from dilute suspensions (initial fluid-content 86 per cent by volume) of cement in various mixtures of ethyl-alcohol (denatured) and water, ranging from all water to all alcohol. One point represents the results obtained in pure toluene.

The addition of alcohol up to about 50 per cent by volume increased the sedimentation-volume;* higher concentrations of alcohol reduced the volume, finally to a point below that for water alone. All suspensions were flocculated to the extent that no separation of particle sizes could be observed, except the one in straight alcohol. In the latter, most of the cement settled out quickly, the largest particles concentrating toward the bottom of the sediment while the very finest flour remained in suspension, even after 24 hours, probably because of Brownian motion.

*Defined as the ratio of bulk volume of the sediment to the solid volume of the particles.

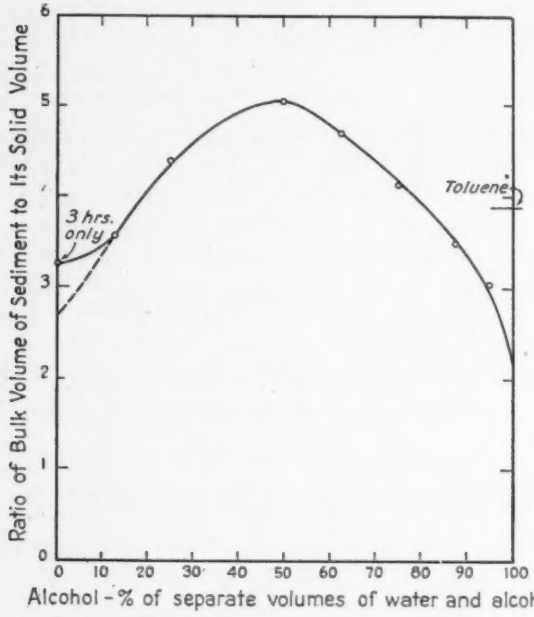

Fig. 1—Sedimentation volumes of cement in alcohol-water solutions and in pure toluene. — Period of settlement 24 hours, except as noted.

Cement 15754

It is reasoned that in settling from dilute suspensions the particle-flocs, in making contact with the sediment, tend to form arches or bridges enclosing relatively large spaces which contribute to the bulkiness of the sediment.[10, 11] The greater the extent to which such arches are able to withstand the pull of gravity, the greater will be the final volume of the sediment. It seems reasonable to assume that the greater the interparticle attraction the greater the strength of the arches and the bulk of the sediment. Therefore, the factors that control interparticle attraction should control sedimentation volume.

The reversal in the slope of the curve of Fig. 1 can be accounted for as follows:* The interparticle force in any given suspension is determined by the kind and concentration of substances in solution and by the force of adhesion between the particles and the liquid medium. Electrolytes in solution or certain types of organic molecules tend to make the particles electrostatically repellent. (Under most circumstances the electrostatic repulsion is at a maximum at very low electrolyte concentrations.) Also, the adhesion of the liquid to the surface and the adsorption of ions or molecules from the solution by the surface tend to cancel some of the particles' surface energy, the surface energy being the source of the force of attraction between the particles. When alcohol is added, it apparently reduces the force of adhesion† between the water and the cement particles

*This explanation is necessarily speculative, since specific data on interparticle forces in this system are lacking. Other explanations can be devised.

†This is directly indicated by the fact that the higher the alcohol content, the lower the dielectric constant of the solution. See Ref. 4.

and thus increases the net force of attraction. At the same time, the addition of alcohol decreases the solubility of the cement constituents and thus favors an increase in electrostatic repulsion, a change tending to offset the effect of the decrease in the force of adhesion between liquid and solid. Evidently, from Fig. 1, one effect is predominant at low alcohol concentration and the other effect is predominant at high concentration.

These data demonstrate that the forces of interparticle attraction in cement-water paste are not as high as they might be and that if a change in the force of flocculation is desired, it could be either an increase or a decrease, according to choice.

Tests with various dispersing agents for portland cement show that they produce a condition similar to that found with cement alone in pure alcohol, but only in very dilute suspensions. Used in concretes or pastes in proportions recommended for field use, they do not cause such dispersion; the pastes clearly show the effects of interparticle attraction. There is evidence, however, that these agents weaken the forces of attraction.

Effect of flocculation on rate of sedimentation

The effect of flocculation on the rate of sedimentation is illustrated in Fig. 2. The curve illustrates the following general rule:

> In *dilute* suspensions, flocculation increases the rate of sedimentation over the average for the dispersed material; in *concentrated* suspensions flocculation *decreases* the rate.

Portland cement pastes as used in practice may be classed as concentrated suspensions; accordingly, *they settle* (*bleed*) *more slowly than they would if the particles were dispersed.*

At high dilution, flocculation increases the rate of sedimentation because the displaced water is able to flow mostly around the flocs, each floc acting much like a single large particle. But at some sufficiently high particle-concentration the particles form a floc-structure so continuous that the displaced water can flow only through the floc-structure itself. In an intermediate range, the flow is partly around and partly through the flocs, a condition giving rise to "channeled bleeding."[13]

The actual conditions in a normal cement paste seem to be about as follows: From considerations already discussed, it seems that before or during the process of flocculation the cement particles become coated with hydrates and the coating acquires a layer of adsorbed water and ions. The particles are so concentrated that when flocculation occurs *they do not draw together into discrete groups* but they form a continuous network. The bonds of the network are, it would be imagined, at points

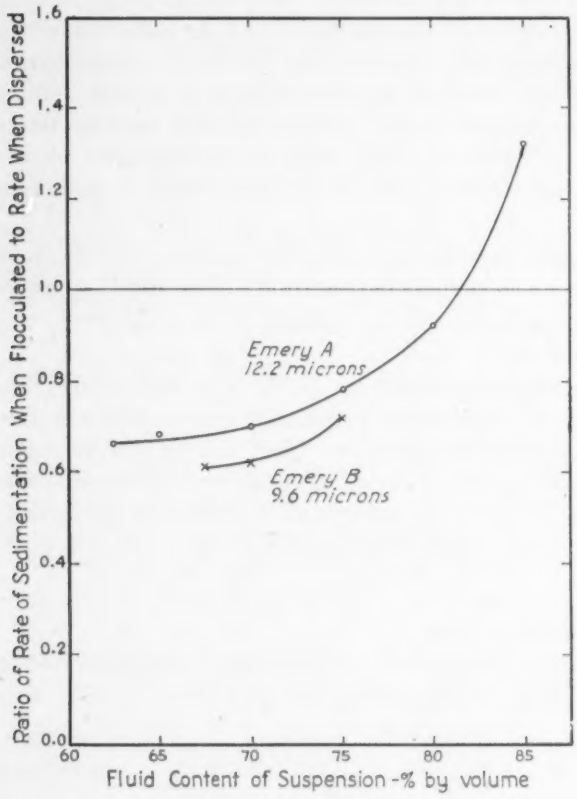

Fig. 2—Effect of flocculation on rate of settlement as influenced by concentration of solids.
(From Ref. 2b)

on adjacent particles that would be in contact were it not for the strongly attached adsorbed layer.

This conclusion is required by at least three conditions:

(1) The nature of the relationship between changes in water content and corresponding changes in the rate of bleeding is such as to show that a change in water content causes a change in the spacing of the network; that is, the effect is *not* that of changing the spacing between clusters of particles.

(2) Steinour's experiments showed that the viscous resistance to sedimentation involves the same amount of particle surface when the particles are flocculated as when they are not flocculated.

(3) Measurements of hydrostatic pressure in cement pastes showed that the particles in a fresh paste are supported entirely by the liquid; that is, none of their weight is transmitted to the bottom of the vessel through point-to-point contacts.

Points (1) and (2) show that the particles cannot exist as separate flocs. Point (3), supported by (1), shows that the particles are normally surrounded with water.

Thus, the evidence is so strong as to be little short of proof that a cement paste is to be regarded as a continuous network of particles in water, the bonds of the network being the forces of flocculation acting across small distances at points of near-contact. In other words, a cement paste may be considered as one large floc; hence, all the water in the paste is within the floc.

So far as the effects of bleeding are concerned the results obtained when the particles are subject to the force of flocculation are clearly preferable to what they are when the particles are free from that force. This becomes doubly evident when it is recalled that during the bleeding of concrete the aggregate soon forms a static framework within the cells of which the paste continues to settle. Even with a normally flocculated paste the settlement is enough to weaken the bond with the undersurfaces of the aggregate particles. Dispersion would not only increase this effect but also would tend to destroy the uniformity of the hardened paste by promoting stratification, as has already been pointed out. If cement pastes were not normally flocculated, it would seem advisable to add a flocculating agent.

From the facts just reviewed it is plain that any claim that *dispersion* is a means of reducing bleeding or "shrinkage before hardening" is based on knowledge of the effect of dispersion on the settlement of *dilute* suspensions and not of the effect on pastes. Also, any deductions based on the assumption that the cement particles in a normal paste exist in discrete flocs from within which water for hydration is excluded are bound to lead to erroneous conclusions, for the evidence is overwhelming that no such condition exists.

Effect of flocculation on plasticity

Flocculation is essential to the plasticity of granular suspensions. When the particles are not flocculated, the mixture behaves like a fluid; it flows under any force, however small, and it can have only the cohesiveness of the liquid itself. When the particles are flocculated, but the dilution is such that individual flocs exist, the mixture probably partakes both of fluidity and plasticity—plasticity increasing with increase in particle concentration. In normal cement pastes, the particles are so concentrated that the behavior of the paste is almost wholly that of a solid; in fact, it meets the definition of a gel, except that it is not colloidal. Under small stresses of short duration it responds both elastically and plastically, by far the greater part of the deformation being plastic.[7] This is true even of pastes thin enough to be poured.

When the strain produced by a sufficiently large force exceeds a limit, the material fails in shear, either with or without dilation, depending on the size and concentration of the particles. Thus when a paste (or a

concrete) is forced through a pipe, the deformation is laminar at first (as it is in fluid flow) until a limiting strain is reached, whereupon the material fails in shear at the pipe-wall, the flow from then on being "plug-flow."[13] With dilute pastes, plug-flow can be converted to viscous flow by producing high rates of flow. With concentrated pastes (or concretes) continuous viscous flow cannot be produced because of dilation due to particle interference.

The fact that a normal paste meets the definition of a solid, even exhibiting some elasticity, seems at first to constitute an anomaly, for already it has been shown that the particles are not in direct contact with each other. However, these facts, put together, only support the conclusion that interparticle attractions are effective across intervening layers of water. It seems self-evident that the cohesiveness, or stickiness, of the paste arises largely from these interparticle forces that give the paste its rigidity.

In contrast to the behavior just described, silica in water, cement in alcohol, or any other suspension in which the particles are not at all flocculated—such mixtures act like fluids; they may be mobile, but they are not plastic, and in the absence of entrained air have only the cohesiveness of the suspending medium. Thus, the question as to whether dispersion would be advantageous raises the question as to whether a fluid paste would be preferable to a plastic one.

It has not been demonstrated that concrete made with a *fluid* paste is more workable than one made with a *plastic* paste. On the face of it, the plastic paste would seem much to be preferred. This is indicated by the undesirable results obtained when pulverized silica, in the absence of a flocculating material such as lime, is used instead of cement. It is indicated also, and very strongly, by the nature of the sediment formed from dispersed suspensions. As said before, the dispersed suspension tends to stratify, but in concrete this is probably less serious than the fact that the sediment formed is very compact and rigid, difficult to redisperse, in contrast to a flocculated sediment which may be only a little less plastic than the original suspension. It is plain that during any delay in transportation or placing, a dispersed paste would exhibit very undesirable characteristics.

EFFECT OF WEAKENING INTERPARTICLE ATTRACTION

The discussion up to this point has dealt mostly with comparisons of the dispersed and flocculated states with respect to the properties of fresh concrete. There remains to be considered the effect of varying the interparticle attraction without producing a state of dispersion.

To interpret the test data that will be presented, we need first to observe that, with given materials, the consistency of fresh concrete depends on two factors:

(1) The quantity of paste
(2) The consistency of the paste

With respect to the properties of fresh concrete, entrained air is considered to be a part of the paste.

Among the materials sold as dispersing agents that we have tested, all changed both the quantity and consistency of the paste in a given mix. One material apparently softened the paste but had little effect on paste volume.* Among those agents that influenced both volume and consistency of paste, all caused air entrainment and some apparently reduced interparticle attraction as did the one that had no effect on paste volume. None of these materials when used in concrete in recommended quantities produced dispersion as defined above. That is, they all left the pastes in the flocculated state, but some of them seemed to reduce the intensity of interparticle attraction. (The statements as to interparticle attraction are made tentative because the evidence concerning interparticle attraction is not quantitative, and is somewhat indirect.) The effects of these different types of agents on the composition and properties of fresh concrete will now be discussed.

Effect of an agent that reduces interparticle attraction without entraining air

The influence of interparticle attraction (force of flocculation) on paste consistency is well illustrated by the effect of oleic acid on a mixture of cement and kerosene. When a paste of cement and kerosene is made, we find that interparticle attraction is so strong that the cement particles form a relatively rigid structure, capable of supporting a small weight. Addition of a very small amount of oleic acid during mixing will produce a noticeable change in appearance and a softening of the paste. This change continues as the dispersing agent is added drop by drop, with constant stirring, the paste becoming much like cement-water paste in consistency and texture. In this state it will bleed like cement paste and have similar plasticity. Finally, a single, last drop of oleic acid (in about 500 cc of paste) will cancel interparticle attraction, i.e., it will produce a state of dispersion.

Thus, we see that varying the concentration of a dispersing agent will change the consistency, cohesiveness and bleeding characteristics of a paste even though actual dispersion is not produced.

The effect of the one agent mentioned above that apparently reduced interparticle attraction without entraining air, here called Agent A, is

*This material is not sold as a dispersing agent. Nevertheless, tests show that when used in sufficient amount, it reduces interparticle attraction.

shown in Table 3. These results indicate that when this agent was present, slightly less paste was required for a given slump.* As a matter of fact, this is the only evidence at hand, other than observations of the behavior of very dilute suspensions, that the interparticle attraction was reduced. We reason that since the average aggregate-particle spacing was less when the agent was present, and since the slump was the same, the paste containing the agent must have been softer. This reasoning arises directly from the observation that the higher the water-cement ratio of the paste, and hence the softer the paste, the less paste required for a given slump.

TABLE 3—INFLUENCE OF AN AGENT WHICH REDUCES INTERPARTICLE ATTRACTION WITHOUT CAUSING AIR ENTRAINMENT—AGENT A

Ref. No. Series 305	Agent % Cem't wt.	Cement Content sks/yd³	Slump in.	Absolute Volume Composition of Concrete				
				Aggreg.	Cement	Water	Air	Water + Air
5	0	4.99	6.6	.748	.088	.150	.014	.164
53	1	4.87	5.6	.756	.086	.139	.019	.158
55	2	4.98	5.9	.762	.088	.138	.013	.151
*	0	4.99	5.6*	.752	.088	.146	.014	.160
2	0	6.00	2.2	.738	.106	.138	.018	.156
54	1	5.98	3.0	.740	.106	.136	.018	.154
56	2	5.94	3.5	.744	.105	.134	.017	.151
*	0	6.00	3.2*	.734	.106	.142	.018	.160

*The figures in this line are estimated from Ref. 5 (or 2) on the basis of a 3 per cent change in water content for a 1-in. change in slump.

From the data in Table 3 it may be deduced that when using this particular agent with these materials, a given water-cement ratio can be obtained with about 5 per cent less cement than the amount required when not using the agent.

From the mere fact that Agent A reduced the water requirement for a given slump, we cannot conclude that workability and other properties were benefited. The net effect of using the agent was to replace a given paste with a smaller quantity of a softer paste. To conclude that this constitutes an improvement in workability would require also the conclusion that lean mixes are more workable than richer ones at the same slump, whereas the fact is that richer mixes are preferable under most conditions because of their greater cohesiveness, greater capacity for plastic deformation,† and greater ability to keep the aggregate from settling while the fresh concrete is in transit. A softening of the paste by reducing interparticle attraction would, in concrete of a given slump, give

*With respect to plasticity, air is considered to be part of the paste as it obviously influences the average spacing of the aggregate particles.

†See Ref. 7b.

relatively less of the desirable characteristics just mentioned. Except in very rich mixes where the paste volume is higher than is necessary for satisfactory workability, the general desire seems to be to enhance these properties by stiffening the paste and increasing its volume. Such is the effect of adding mineral powders or increasing the cement content.

In this connection it may be recalled that a cement of high specific surface makes a stiffer, more cohesive paste than does a coarser one at the same water-cement ratio. Yet experience shows that only in very rich mixes is it necessary to use more water (or more paste) when a cement of high specific surface is substituted for a coarser one. This indicates that under some circumstances a stiffening, rather than a softening, of the paste is advantageous. An extreme example of this was reported by Kennedy.[14] He found that although no amount of water would make a sand-and-gravel mixture plastic, air entrained in the mixing water with a suitable agent made the mix appear as if it had been made with cement paste. As will be shown below, entrained air has a stiffening effect.

In general, it seems that increasing the stiffness of the paste, that is, giving it more "body," appears to be advantageous under ordinary conditions in mixes containing less than about 5½ sacks of cement per cu. yd. (1½-in. maximum size aggregate) or, in other terms, it appears advantageous if w/c exceeds about 0.5 by weight.

The lack of benefit from weakening the force of flocculation is probably due to the fact that the cement particles are normally not very strongly flocculated. This was pointed out before in connection with Fig. 1. It seems likely that if cement in water were flocculated as strongly as is cement in kerosene, a reduction in interparticle attraction might be beneficial under most, if not all, circumstances. But, in view of the relative weakness of the flocculating forces in cement-water paste, the value of further reductions in interparticle attraction is debatable.

This point may be emphasized by considering again the effect of reducing interparticle attraction on bleeding characteristics. The effect is to *increase* the amount of settlement (bleeding). However, the *rate* of settlement is affected very little. (See Table 2). The fact that the rate of settlement is affected very little shows that the *initial* texture of the paste is virtually unaffected by reducing the intensity of interparticle attraction. However, the *final* texture, after settlement, would be affected, the pastes having the lowest interparticle attraction forming the densest sediment. This might be advantageous were it not for the fact that any increase in the amount of paste settlement is accompanied by an increase in the depth of the under-aggregate fissures, which fissures weaken the concrete and make it more permeable. To be considered also is the probability that weakening the force of interparticle attraction

increases the tendency toward channeled bleeding with its attendant undesirable "sand boils"* and it decreases the ability of the paste to hold the concrete in a plastic state while the concrete is standing or in transit. All effects considered, it seems very doubtful that a weakening of the force of flocculation improves the properties of fresh concrete, even though the slump may thereby be increased, except perhaps in mixes unusually rich in cement.

Effect of an air-entraining agent having little or no effect on interparticle attraction—Agent B

The use of some agents causes a pronounced increase in the air content of cement paste or concrete without appreciable effect on interparticle attraction. The manner in which this is brought about may be explained in simplified version as follows:[15] Any mechanical process that mixes a liquid with a gas tends to form a foam, although the foam may be scant and its life extremely short. If an agent is added that lowers the surface tension of the liquid (a "capillary-active" material), a foam forms more easily and it usually lasts longer than it does without the agent, especially if the agent, by reason of its adsorption at the air-liquid interface, enables the films to withstand shocks. Many organic compounds have this effect when used with water, the soaps being perhaps the most common class. A molecule of soap is relatively large, of the long-chain variety, having a hydrophilic "head" and a hydrophobic "tail." Such molecules tend to collect at the boundary between water and air (or water and oil) and, according to theory, array themselves in such a way that the hydrophilic part remains in the water and the less wettable (hydrophobic) remains in the air. These molecules thus create a boundary film which lowers the surface tension of the water and stabilizes any foam that is formed by mechanical action.

When such materials are used with portland cement paste, the entrained air is probably not to be regarded as a foam, strictly speaking. But the individual bubbles scattered through the paste are no doubt stabilized by the same mechanism that stabilizes the foam; therefore, it is appropriate to speak of such materials as foam stabilizers or air-entraining agents.

Fig. 3 and 4 show the effect of an air-entraining agent. In each diagram the lower line shows how the amount of water decreases as the air content increases. The upper line represents the sum of the volumes of air and water. Since all the mixes represented in a given diagram had the same cement content, the rise of the upper line represents the increase in paste content and the corresponding decrease in aggregate

*In normal bleeding, water flows to the surface uniformly through all the interparticle spaces. In "channeled" bleeding, some of the flow occurs through randomly spaced channels which are the result of ruptures in the normally continuous "mesh" of the flocculated paste (see Ref. 12). The probability of the occurrence of such ruptures increases as the floc-structure is weakened by dilution. Presumably, the probability would be increased also by weakening interparticle attraction by means of a dispersing agent.

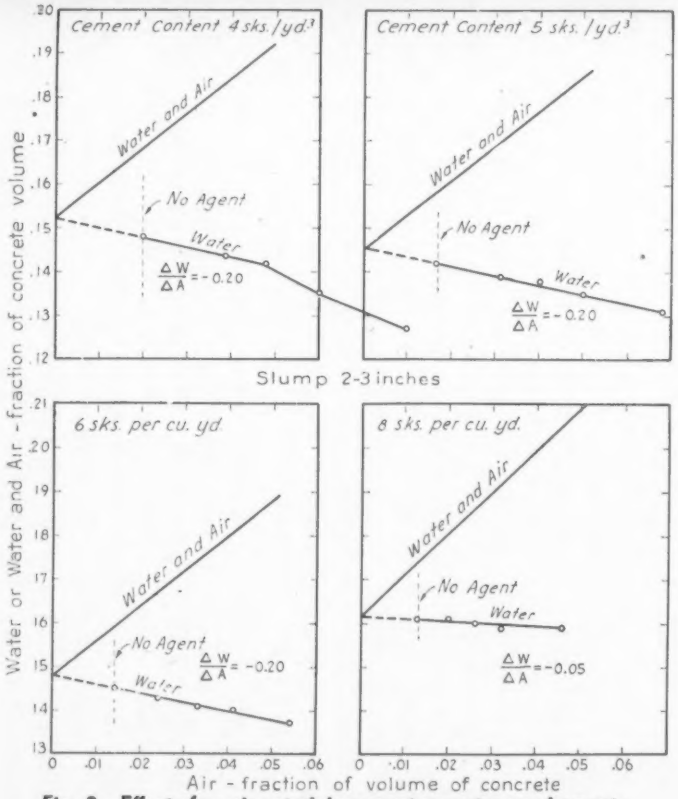

Fig. 3—Effect of an air-entraining agent on water-requirement

Cement: a mixture of four Type I commercial cements.

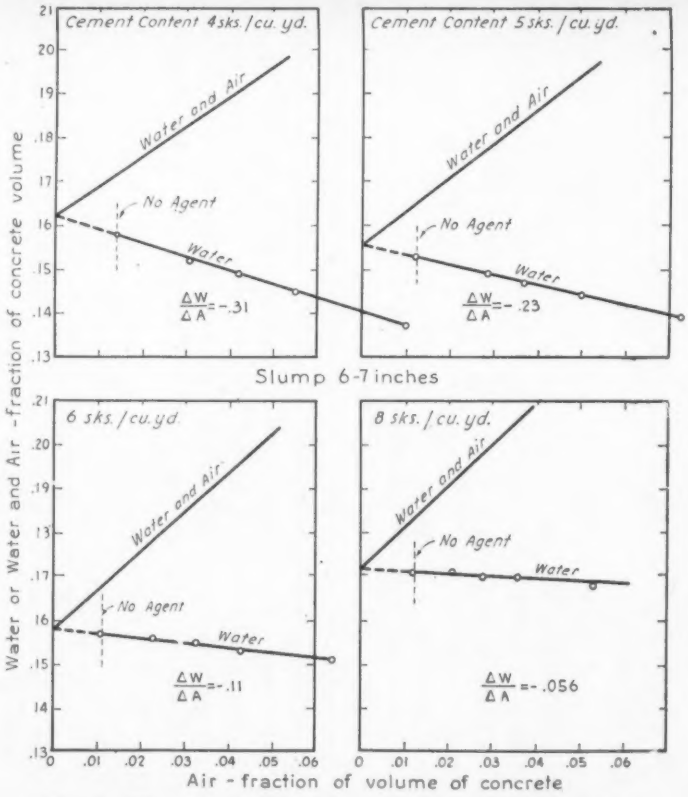

Fig. 4—Effect of an air-entraining agent on water-requirement

Aggregate: Elgin sand and gravel, max. size 1 in.

content. Figs. 3 and 4 represent concretes of different consistencies, as stated in the titles.

In every case the increase in air content was accompanied by a decrease in water content, the decrease being directly proportional to the increase in air content. But since the decrease in water was not as great as the increase in air, the paste content also increased in direct proportion to the increase in air content. If air and water had the same effect on slump, the slope of the water curve in each diagram would have been −1.0; i.e., each per cent of air would have replaced exactly 1 per cent of water and the paste content would have remained constant. But, as shown in the diagrams, each 1 per cent of air replaced only 0.3 per cent or less, and the paste content increased. Since the paste required for a given slump increases in direct proportion to the increase in air content, we must conclude that entrained air stiffens the paste. It follows that when the water content is not reduced, the increase in slump caused by entraining air in a given mix is due to the increase in paste content and not to a softening of the paste.

The conclusion that the paste is stiffened by entrained air is in line with the observed effect of whipping air into cream, or the stiffening that is produced when liquid-liquid emulsions are formed. Indeed, the stiffening effect of entrained air on cement paste was observed directly by Kennedy.[14]

Some idea as to the degree to which entrained air stiffens the paste was deduced from the data in Fig. 3. As a first step in this deduction, Fig. 5 was prepared. The lower diagram shows the relationship between water or water + air content and the cement content for mixes containing definite amounts of air, the curves being obtained by interpolation in the diagrams of Fig. 3. The positions of the lines in the upper diagram in Fig. 5 were then obtained from the lower diagram by adding the volume of the cement to that of water + air. The next step was to assume, as before, that at equal paste volumes and equal concrete slumps the pastes must have the same consistency, and then, by means of Fig. 5, to compare the composition of pastes of the same consistency with and without entrained air. The results for two paste contents are given in Table 4.

These figures show, for example, that a paste for which w/c = 0.583 and air/c = 0.12 had the same consistency as one for which w/c = 0.447 and air/c = 0. Thus, introducing 0.12 cc of air per g of cement stiffened the paste as much as would reducing w/c from 0.583 to 0.447, i.e., reducing the water 0.136 cc per g of cement. It is not likely that pastes of these compositions would show exactly the same consistency if tested separately from the concrete, for, as indicated by Kennedy's experiments, effectiveness of the air is probably influenced by the aggregate. Never-

TABLE 4

Air Content of Concrete	w/c		$\frac{\text{Air} + \text{w}}{\text{c}}$	$\frac{\text{Air}}{\text{c}}$
	Abs. Vol.	Wt.	cc per g	cc per g
Paste = 0.25				
0.00	1.42	.447	.447	0
0.01	1.46	.460	.495	.035
0.02	1.54	.485	.555	.070
0.03	1.85	.583	.703	.120
Paste = 0.27				
0.00	1.31	.413	.413	0
0.01	1.33	.419	.448	.019
0.02	1.36	.429	.488	.059
0.03	1.38	.434	.531	.107
0.04	1.40	.441	.575	.134
0.05	1.66	.523	.718	.195

theless, the data show conclusively that entrained air has a stiffening effect even though it is more fluid (i.e., it has a lower viscosity) than the water it displaces. A correct explanation of the effect would undoubtedly involve the surface tension at the numerous air-water interfaces.

Returning to Fig. 3 and 4, we may note that the point for the plain mix in each diagram falls in line with the other points. This indicates that the agent had no softening effect on the paste, that is, it did not reduce interparticle attraction. Had there been a reduction in interparticle attraction the line would have passed below the point for the plain mix.

In general it appears that the lower the cement content the greater the reduction in water requirement per unit increase in air content. This is shown as a steady trend in Fig. 4, but in Fig. 3, representing the stiffer consistency, the effect is equal in the 4-, 5-, and 6-sack mixes.

So far as workability and bleeding characteristics are concerned, the effect of entrained air may be regarded as highly beneficial. It increases cohesiveness, it aids in holding the aggregate from settling while the concrete is being transported, and the increase in paste volume increases the capacity for plastic deformation.

Entrained air reduces strength, but greatly increases resistance to frost action.[16]

Effect of an agent that both reduces interparticle attraction and entrains air—Agent C

The effect of an agent of this type on water requirement and air content is shown in Fig. 6. The data are fewer than might be desired,* but

*Although many tests have been made on various agents in this laboratory, only a few have included several proportions of a given agent.

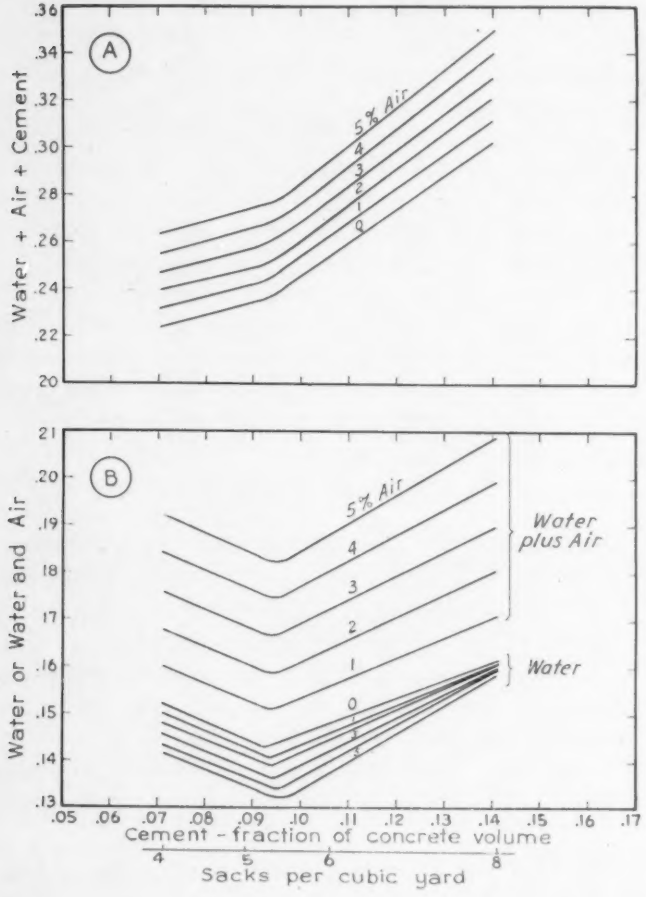

Fig. 5—Effect of an air-entraining agent on water, air, and paste content—slump 2-3 in.

their indications are such as would be expected from the discussion of the two other types presented above. In Fig. 6 the solid lines represent the water contents and water + air contents as in Fig. 3 and 4. The broken line is drawn to have the same slope as the lines in Fig. 3 or 4, for corresponding slumps and cement contents, and to pass through the point representing the plain mix, except where adjustments were made to compensate for inequalities in slump. Thus the broken line represents the effect of an air-entraining agent, Agent B, and the solid lines, the effect of the agent which both reduces interparticle attraction and entrains air, Agent C. The triangular point in each diagram represents the one mix of each kind in this series that was made with Agent B.

Although the four diagrams are not wholly consistent with each other in all respects, they show that Agent C reduced the water requirement more than did Agent B. Presumably, this is the effect of a reduction in

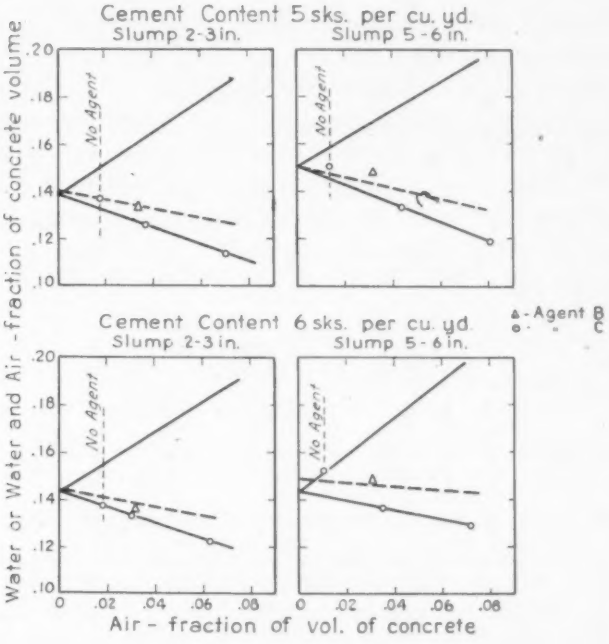

Fig. 6—Effect of an agent that entrains air and reduces interparticle attraction.

Cement: a mixture of four Type I commercial cements. Aggregate: Elgin sand and gravel, max. size 1½ in.

interparticle attraction caused by Agent C, as indicated by points for the plain mixes falling above the line for Agent C.

It is clear, however, that even if Agent C did reduce the interparticle attraction, the net effect of this and the air entrainment was a stiffening of the paste. This is shown by the fact that with Agent C, as with Agent B, the paste content required for a given slump increased in direct proportion to the increase in air content. In view of this, it may be concluded that whatever undesirable effects on plasticity and bleeding characteristics the reduction in interparticle attraction might have, they are offset by the effect of the entrained air.

On the whole, it appears that *with respect to the properties of fresh concrete*, agents like B and C are beneficial, C being somewhat more so, for with an agent of this type, concrete of a given slump and air content can be obtained with less water in the concrete.

Whether or not the properties of the *hardened* concrete are benefited equally, or at all, by agents of this type is a question that cannot be dealt with in general terms. Experience indicates that some agents have no effect on the chemical processes of hardening and therefore the effects on strength can be predicted fairly well from the effect on the voids-cement ratio. Other agents, especially those that influence interparticle attraction, are liable to retard hydration and, apparently for that reason, some

are sold with an added accelerator. Agents of this type have different effects on different cements.

With respect to durability, entrained air is decidedly beneficial, and therefore the use of agents like B and C can be recommended where special protection against frost action is necessary. The beneficial effect of entrained air on durability is probably the result of providing room for expansion of water during the process of freezing.[16b] It is difficult to see how either dispersion, per se, or a weakening of the forces of flocculation in the absence of air entrainment, could have much influence on frost resistance.

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Discussion of a paper by T. C. Powers:

Should Portland Cement Be Dispersed?*

By E. W. SCRIPTURE, JR., FRED M. ERNSBERGER
and WESLEY G. FRANCE, HOWARD R. STALEY,
C. A. G. WEYMOUTH, M. SPINDEL, EMIL SCHMID,
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and AUTHOR

By E. W. SCRIPTURE, JR.†

This paper attacks a problem of considerable interest in cement technology but is somewhat confusing and in some instances contradictory. In the first, theoretical part of the paper, it appears to conclude on the basis of certain well-known principles of the behavior of *dilute suspensions*, plus a number of assumptions which are not supported by experimental data, that dispersion of cement in cement pastes, i. e., in *concentrated suspensions*, would be undesirable because it would increase bleeding, increase permeability, decrease bond strength, weaken the concrete and cause segregation in transit and on standing.

The second part of the paper, which includes a limited number of experimental data on three materials sold for addition to concrete mixes, seems to conclude, on the basis of the preceding argument and the limited experimental data that the materials which are stated to be cement dispersing agents by the manufacturers do not in fact disperse cement in cement pastes; that one material increases bleeding, lowers bond strength, produces fissures, weakens the concrete, increases permeability, and increases the tendency toward segregation; that a second material, an air entraining agent but not a dispersing agent, has beneficial effects with respect to water reduction, workability, bleeding and durability, although it reduces strength, and that a third material which the manufacturers call a dispersing agent and which also entrains some air, does not disperse cement in cement pastes but has similar beneficial effects to the second material but in a higher degree. No comment is made on the third material with respect to its effect on strength.

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The net result of this rather perplexing line of reasoning appears to be to conclude that cement dispersion does not exist in cement pastes, that if it did it would be undesirable, but that a material which contains a cement dispersing agent (at least it disperses cement in dilute suspensions) and also has some air entraining effect, has beneficial effects in concrete, greater than those of an air entraining agent per se. It is difficult to reconcile the conclusion that cement dispersion (or reduction in interparticle attraction) plus air entrainment has a more beneficial action than an equal or greater air entrainment alone with the conclusion that cement dispersion does not exist and that if it did the effects would be harmful.

The essence of the paper does not seem to be to show that materials which are sold as cement dispersing agents are not beneficial in concrete or are harmful. On the contrary, the paper specifically sets forth the beneficial effects of one such material and even recommends its use where resistance to frost action is desired. The paper does seem to attempt to discredit the explanation of any beneficial effects on the basis of cement dispersion and to attribute them solely to air entrainment although no explanation is offered on the better performance of Agent C, a cement dispersing agent plus air entrainment compared with Agent B, which is an air entraining agent only. This attempt to discredit cement dispersion as a factor in concrete mixes is based on an arbitrary definition and a process of reasoning from well-known phenomena obtainable in dilute suspensions and assumptions not supported by experimental evidence.

In view of the interest which cement dispersion has aroused in some quarters it may be worthwhile to attempt to analyze in somewhat greater detail the reasoning and evidence in this paper. For this purpose the paper may be considered in three parts, first the enunciation of principles of colloid chemistry, the definition of dispersion and the assumptions made, second the conclusions reached without experimental evidence on the basis of the preceding, and third the data and conclusions on three commercial materials. In making this analysis the important statements (arabic numerals) in each part of the paper are quoted, followed by a discussion.

1. PRINCIPLES AND ASSUMPTIONS

a. Wetting

1. If the solid and the liquid show a strong mutual attraction considerably greater than the surface tension of the liquid, the liquid will spread over the solid surface without outside aid.

2. The attraction between cement and water is so strong that each cement grain becomes completely surrounded by water even though in dilute suspension the grains are clustered.

The first statement is undoubtedly a correct statement of a basic principle which defines the conditions under which a liquid (water) will spread on a solid surface (cement) when it is brought in contact with that solid surface and is free to spread without obstacles. The second statement is an assumption not supported by experimental evidence. If the cement is normally flocculated, tending to hang together in clusters (clumps), then in order for the water to break up these clusters and wet completely the entire surface area of all the particles it is necessary for the attraction between the cement and water to overcome not only the surface tension of the water but also the force of attraction between the cement particles. This is not necessarily a condition which is fulfilled in the case of cement and water, and anyone who has had experience with clays, also very hydrophilic solids, and the difficulty of making clay slips with water, will be led to the conclusion that complete wetting of cement by water is hardly probable.

b. Dispersion

1. Interparticle repulsion is not necessary for dispersion.

2. Portland cement cannot be caused to disperse spontaneously for cement particles are predominantly microscopic, not colloidal.

3. Some of the phenomena pictured in connection with the use of dispersing agents with portland cement actually can occur only among particles that are of truly colloidal dimensions.

The first statement is correct if applied to particles of colloidal dimensions where the force of Brownian movement is sufficient to disperse particles having no mutual attraction (but no mutual repulsion, or mutual attraction of a very low order). It is not, however, especially pertinent since the real problem is whether a finely divided solid can be maintained in a dispersed state in a liquid. A solid or a liquid can be dispersed mechanically but whether it remains dispersed depends on the means used for stabilization. Interparticle repulsion is not the only means of stabilizing the dispersion but it is one of the most effective. The second statement is entirely correct if it is stated, as seems to be intended, that portland cement will not disperse spontaneously but requires a dispersing agent for this purpose. The third statement is vague in that it does not specify what phenomena are not applicable to portland cement. If it means that Brownian movement does not occur with the major part of the cement particles it is correct, although some of the cement particles are subject to this phenomenon when dispersed in water. Other than that it is not apparent what this statement means. That portland cement particles, whether of colloidal or microscopic size, can be dispersed in dilute suspensions by means of a suitable dispersing agent can readily be demonstrated.

c. Interparticle attraction and paste properties

1. The greater the interparticle attraction, the stiffer a paste will seem to be when it is stirred.

2. If the forces of repulsion predominate (i. e., if the cement is dispersed) particles that would remain in contact when quiescent become separated as they fall through the liquid during sedimentation.

3. When the attractive forces are very weak, and especially when the particles are mutually repellent (dispersed), the sediment that is formed tends to be non-uniform in composition, the proportion of coarse particles increasing toward the bottom of the sediment.

The first statement is entirely correct and merely confirms that cement dispersion gives greater fluidity of the mix with a given water-cement ratio. The second statement is also correct but unduly limited since, when the forces of repulsion predominate the particles become separated whether they are falling through the liquid during sedimentation or not. The third statement is quite correct also, if it is qualified by saying that this observation applies to dilute suspensions, not cement pastes. The writer's work published some twenty years ago on the determination of particle size distribution of clays by sedimentation illustrates this phenomenon.[1] Later in the paper it is correctly stated: "Also in *dilute* suspensions of particles, segregation of sizes takes place during sedimentation if the interparticle attraction is absent or weak and it does not take place if interparticle attraction is strong." With respect to cement pastes, which are highly concentrated suspension, the statement is an assumption in support of which no data are adduced, It is confusing when included in a paragraph on paste properties.

d. Definition of dispersion applicable to cement paste

1. When interparticle attraction in a fresh cement paste is so weak that it has no appreciable effect on the behavior and physical properties of the paste, the particles in the paste may be said to be dispersed.

2. It should be noted that the definition does not rest on the presence or absence of particle clusters.

3. The reader is asked to avoid applying the final conclusions concerning dispersion as defined her, to dispersion defined in some other way.

An arbitrary definition of dispersion is here established which has absolutely no meaning and is not consistent with scientific and industrial use of the term. Any definition which includes the term "appreciable" is quantitatively useless. In order to apply the definition it is necessary to know whether interparticle attraction appreciably affects the behavior and physical properties of the paste. No criteria are given for telling whether the behavior and physical properties of the paste are or are not affected by the existing interparticle attraction in any

[1] Edward Schramm and E. W. Scripture, Jr.: The Particle Analysis of Clays by Sedimentation, *J. Am. Ceram. Soc.*, V. 8, p. 243-252.
The Particle Size Distribution of Typical Feldspars and Flints, *J. Am. Ceram. Soc.*, V. 10, (April 1927).

given case. Hence it is impossible to apply this definition and impossible to determine in any given case whether the cement is or is not dispersed in accordance with this definition. It is suggested that when interparticle attraction is absent or negligible (dispersed) a suspension that is not too concentrated flows like a true liquid, but when interparticle attraction is not negligible, the suspension acquires the properties of a plastic solid to some degree. The fallacy in this as a criterion for dispersion is that at some concentration the suspension will flow like a liquid whether it is dispersed or flocculated and that at some other concentration it will have the properties of a plastic solid to some degree whether dispersed or flocculated. The commonly accepted scientific and technical meanings of the terms flocculated and deflocculated are, in the first instance that the particles form clusters and in the second that they do not, yet the second statement given seems to make a virtue of discarding all previous concepts of dispersion and flocculation. The third statement simply emphasizes that, since all accepted definitions of dispersion have been abandoned, anything which may be said or concluded is not applicable to materials which deflocculate or disperse cement in accordance with accepted definitions of this phenomenon.

e. Dispersion of portland cement

1. There is no question but that cement particles in a normal paste are flocculated.
2. The electrolytes (apparently the hydroxyl ions) bring about flocculation of the cement particles.
3. During this period (the first five minutes after addition of water) a coating of hydrates forms on the cement grains.

The first statement simply confirms what is well-known, that the normal state of portland cement in water is the flocculated state. The second statement may be correct but no evidence is adduced to show that it is. A later statement "Electrolytes in solution or certain types of organic molecules tend to make the particles electrostatically repellent" seems to contradict it. Experimental evidence[2] with clays, flints, feldspars, and similar materials which show that hydroxyl ions exert a deflocculating effect also indicates that this explanation of the flocculation of cement is not correct. It seems much more probable that the cement is naturally flocculated in the original state and becomes slightly less so during the first five minutes. The third statement is probably correct, although unsupported, but is rather curious in that it seems to negative the statement previously made that the entire cement surface was wetted and free to hydrate. Even if it is assumed, and the assumption is not supported by evidence, that all the cement

[2] E. W. Scripture, Jr., and E. Schramm: The Deflocculation of Clay Slips and Related Properties, *J. Am. Ceram Soc.*, V. 9, p. 175-178 (April 1926).
E. W. Scripture, Jr.: The Behavior of Feldspars and Flints with Acids and Bases, *J. Am. Ceram. Soc.*, V. 10, p. 238-242 (April 1927).

grains are completely wetted when water is added, it appears highly probable that, in the flocculated condition, these hydrate coatings would be in contact with one another and would preclude or at least impede further hydration at the points of contact.

2. CONCLUSIONS

a. Effect of flocculation on amount of settlement

1. The data demonstrate that the forces of interparticle attraction in cement-water paste are not as high as they might be and that if a change in the force of flocculation is desired, it could be either an increase or a decrease according to choice.
2. Used in concrete or pastes in proportions recommended for field use, they (various dispersing agents for portland cement) do not cause much dispersion; the pastes clearly show the effects of interparticle attraction.

This first statement is probably correct but is certainly not derivable from the data given. All these relate to dilute suspensions, mainly suspensions of solids other than portland cement in water. Such data cannot justify any conclusions regarding cement pastes which are concentrated suspensions of portland cement in water with respect to the second statement. It has been shown by Ernsberger and France[3] that calcium lignosulfonate is adsorbed by portland cement and endows the particles with electrostatic charges making them mutually repellent in dilute suspensions. This is clearly demonstrable by microscopic observation and the increase in fluidity of concretes and mortars for a given water-cement ratio with such an agent, even when no additional air is incorporated in the mix, shows that this dispersion effect is carried over from dilute suspensions to pastes to a very appreciable extent. Whether the cement particles in a paste (concentrated suspension) can be said to be dispersed is largely a matter of definition since they obviously cannot be widely separated although they may still be mutually repellent.

b. Effect of flocculation on rate of sedimentation

1. So far as the effects of bleeding are concerned the results obtained when the particles are subject to the force of flocculation are clearly preferable to what they are when the particles are free from that force.
2. It is plain that any claim that dispersion is a means of reducing bleeding or "shrinkage before hardening" is based on knowledge of the effect of dispersion on the settlement of dilute suspensions and not of the effect on pastes.
3. Also, any deductions based on the assumption that the cement particles in a normal paste exist in discrete flocs from within which water for hydration is excluded are bound to lead to erroneous conclusions for the evidence is overwhelming that no such condition exists.
4. Dispersion would not only increase this effect (weakening of bond with the undersurfaces of aggregate particles) but also would tend to destroy the uniformity of the hardened paste by promoting stratification.

[3] Fred M. Ernsberger and Wesley G. France: Portland Cement Dispersion by Adsorption of Calcium Lignosulphonate, *Ind. and Eng. Chem.*, V. 37, p. 598-600 (June 1945).

5. If cement pastes were not normally flocculated, it would seem advisable to add a flocculating agent.

For these conclusions there is very little evidence. In addition to the previous statements of principles and assumptions the only experimental evidence is some data on emery suspensions of 12.2 mu and 9.6 mu particle sizes which do not bear any necessary relation to portland cement pastes. It seems desirable to take each conclusion up individually:

It is by no means clear that, for concentrated suspensions such as pastes, the flocculated state is preferable to the dispersed state with respect to bleeding. Actually "bleeding", as the term is commonly used in concrete technology, is not a phenomenon which occurs in dilute suspensions but this conclusion is based on reasoning from observation on dilute suspensions. It has been demonstrated that bleeding is greatly reduced and in many cases practically eliminated with a cement dispersing agent. The author might, of course, attribute this to air entrainment, as appears later in the paper, but this is not consistent with the fact that the dispersing agent exhibits a greater reduction in bleeding than do air entraining agents which do not have a dispersing effect on cement but do entrain much more air. Some data on this point are shown in Table A.

TABLE A

Addition	Percent bleeding Concrete Mix—6.0 sks. cu. yd.		
	Cement I	Cement II	Added air—percent
None	100	100	0.0
Air entraining agent A	42	34	3.4
Air entraining agent B	49	49	3.3
Dispersing agent	25	20	2.9

It is obvious that a reduction in bleeding of 77½ percent with the dispersing agent cannot be solely attributable to air entrainment since the same mixes with higher air contents give only 62 percent and 51 percent reductions. It is a logical conclusion that the additional reduction in bleeding is attributable to cement dispersion.

Much the same remarks apply here as were made in connection with the first statements. That claims for cement dispersion are based only on a knowledge of the behavior of cement in dilute suspensions is an unwarranted statement; the writer's work on cement dispersion has included dilute suspensions but studies of bleeding and "shrinkage before hardening" have been extensive and necessarily conducted on pastes, mortars, and concretes. The writer has, more-

over, carried on extended investigations of dispersion and related phenomena with both dilute and concentrated suspensions of clays and other ceramic materials which behave in many respects in a similar manner to cement. Any claims for reduction in bleeding and "shrinkage before hardening" would be based on a knowledge of behavior of concentrated not dilute suspensions. The behavior of dilute suspensions would lead to the conclusion stated in the paper, that bleeding would be increased, but this does not imply that it would be in concentrated suspensions. In drawing this conclusion the author does just what he says should not be done, in applying reasoning from observations on dilute suspensions to cement pastes.

Certainly discrete flocs exist in dilute suspensions as can be shown by microscopic observation. If there is overwhelming evidence that such flocs disappear when the suspension is concentrated it is not given. That these flocs coalesce as the suspension becomes more concentrated until in a paste there is a continuous network of flocs does not alter the fundamental condition that the paste is basically made up of discrete flocs. As an illustration, sand distributed sparsely on the floor exists in the form of discrete particles; that these grains of sand are all in contact with one another when this sand is gathered together in a box does not alter the fact that the sand is made up of discrete particles. That water for hydration is rigidly excluded from the flocs is very improbable; it does appear logical that hydration would be impeded where the particles making up the flocs are in contact and that water trapped within the flocs would not contribute to the fluidity of the paste.

This is borne out by the well-established increase in fluidity of a mortar or concrete when a dispersing agent is added, even without additional air. If this explanation is incorrect, the author fails to offer any other for this phenomenon which he admits exists.

The fourth conclusion is unsupported by evidence. The author again commits the error in reasoning which he deplores, the drawing of conclusions with respect to cement pastes from the behavior of dilute suspensions. As previously noted, a dispersing agent reduced bleeding to a greater extent than is secured by air entrainment, and observation of countless mortars and concretes over a period of years has failed to disclose any indication of stratification.

The fifth is again a totally unsupported conclusion based on assumptions. It is further contradicted by the author himself later in the paper when he says "on the whole it appears that with *respect to the properties of fresh concrete*, agents like B and C (a dispersing agent) are beneficial, C being somewhat more so, for with an agent of this type, concrete of a given slump and air content can be obtained

with less water in the concrete." Where the author has made any actual tests he finds cement dispersion beneficial; where he departs into the realms of theoretical reasoning and assumptions he finds it harmful.

c. Effect of flocculation on plasticity

1. Flocculation is essential to the plasticity of granular suspensions.

2. It seems self-evident that the cohesiveness or stickiness of the paste arises largely from these interparticle forces that give the paste its rigidity.

3. It has not been demonstrated that concrete made with a fluid paste is more workable than one made with a plastic paste.

4. It is plain that during any delay in transportation or placing a dispersed paste would exhibit very undesirable characteristics.

Most of these conclusions are unsupported by experimental evidence and are based on observations of the behavior of dilute suspensions, a line of reasoning which the author has already vehemently decried. The first is a sweeping generalization which is manifestly incorrect. Dilute suspensions will not be plastic whether the suspended granular solid is flocculated or dispersed. Concentrated suspensions may have plastic properties even though the particles are mutually repellent, i. e., dispersed; provided that they have suitable shape, surface characteristics, and water adsorption properties. This is true of cement as is readily demonstrable. It is equally true and even more readily demonstrable with clays. Some clays are naturally flocculated; others exist naturally in the dispersed state; both form plastic pastes with water. That dispersion alters the plastic characteristics of cement and other pastes is self-evident but that it destroys the plasticity is not a tenable proposition. It is of interest to note that Lewis, Squires, and Broughton[4] state that "Dispersion of the clays is essential to plasticity". The second conclusion appears fairly reasonable in itself but neglects the considerations applicable to concentrated suspensions such as pastes. Although the particles may be electrostatically charged and mutually repellent, in concentrated suspensions interparticle forces still play a part and also the forces of adsorption which tend to produce a condensed layer of water around each particle are factors which must be given consideration. The third conclusion is totally meaningless. It might equally well be said, with as much foundation, that it has not been demonstrated that concrete made with a stiff paste is more workable than one with a fluid paste. Either statement involves a definition of workability which is an indefinite term varying in meaning depending on the application. Workability merely means that the material has suitable fluid and/or plastic properties for the use to

[4] Lewis, Squires, and Broughton: Industrial Chemistry of Colloidal and Amorphous Materials, The MacMillan Co. 1942, p. 455.

which it is to be put. Nothing could be less plain than the last unsupported conclusion. The experience of readymix companies, as well as other users of dispersing agents in concrete over a number of years, shows conclusively that, with some dispersing agents at least, concrete in which a dispersing agent has been used exhibits highly desirable characteristics during delays in transportation and placing[5].

d. Effect of weakening interparticle attraction

1. None of these materials when used in concrete in recommended quantities produced dispersion as defined above. That is, they all left the pastes in the flocculated state, but some of them seemed to reduce the intensity of interparticle attraction.
2. The statements as to interparticle attraction are made tentative because *the evidence* concerning interparticle attraction *is not quantitive and is somewhat indirect.*

These two statements just about answer each other. The author is not warranted in his conclusion that the materials of which he is writing do not produce dispersion simply because, as he says in his next sentence, he has neither quantitative nor direct evidence on the interparticle forces existing in the pastes. Further, he has used as a criterion of the existence or non-existence of dispersion a definition of dispersion which he has invented for himself and which does not accord with the commonly accepted meaning of the term.

It is very difficult to discuss the results of the tests given in the paper and the conclusions drawn therefrom since the author does not name the agents used or even give their chemical compositions. The reader is at a distinct disadvantage in trying to apply the author's conclusions in a particular case or to cement dispersion generally. From the general descriptions given an attempt is made to examine the author's conclusions as they relate to the writer's experience with certain dispersing agents, especially those of the lignosulphonic acid type.

3. EFFECT OF THREE AGENTS ON CEMENTS

a. Agent A

1. From the data it may be deduced that when using this particular agent with these materials a given water-cement ratio can be obtained with about 5 percent less cement than the amount required when not using the agent.
2. The net effect of using the agent was to replace a given paste with a smaller quantity of softer paste.
3. The lack of benefit from weakening the force of flocculation is probably due to the fact that the cement particles are normally not very strongly flocculated. The value of further reduction in interparticle attraction is debatable.
4. The effect is to increase the amount of settlement bleeding.
5. This might be advntageous were it not for the fact that any increase in the amount of paste settlement is accompanied by an increase in the depth of the under-aggregate fissures, which fissures weaken the concrete and make it more permeable.
6. To be considered also is the possibility that weakening the force of interparticle attraction increases the tendency toward channeled bleeding with its attendant undesir-

[5] R. E. Hickson, "Placing a Heavy Concrete Terminal on the South Jetty of the Columbia River," *Pacific Builder and Engineer*, p. 41-44 (April 1944).

able "sand boils" and it decreases the ability of the paste to hold the cements in a plastic state while the concrete is standing or in transit.

The author does not appear to be sure whether this agent is or is not a dispersing agent but states in a footnote that "This material is not sold as a dispersing agent, nevertheless tests show that when used in sufficient amount it reduces interparticle attraction". From data on a product which might or might not be a dispersing agent, and probably is not, it seems hardly reasonable to draw any general conclusions regarding the effect of dispersion of the cement on concrete. What is said under Agent A may be correct with respect to Agent A but it is desired to point out that these conclusions are not applicable to cement dispersing agents other than Agent A, even if it is assumed that Agent A is a dispersing agent.

Increased Bleeding—Bleeding is reduced with a dispersing agent.

TABLE B

Addition	Percent bleeding Concrete mix—6.0 sks./cu. yd.	
	Cement I	Cement II
None	100	100
Dispersing agent	25	20

TABLE C

Concrete sks./cu. yd.	Percent reduction in bleeding with dispersing agent
4½	60
6	55

Weakened Concrete—Compressive and flexural strengths are increased with a dispersing agent. One example[6] from a large mass of available data on compressive and flexural strengths is shown in Fig. A.

TABLE D

Addition	Permeability*
None	11.3
Admix B (corresponds to Agent C)	7.6
Admix C (corresponds to Agent A)	6.5

*Liquid inflow in gals. per 1000 sq. ft. per 24 hrs.

[6] Report of Tests, Smith-Emery Company, Los Angeles, Cal. — Consolidated Vultee Aircraft — Lindbergh Field Project, 1944.

Increased Permeability—Concrete is made less permeable with a dispersing agent. Reference is made to one published paper[7] and to a published study[8] on the penetration of kerosene through concrete. In this

[7] W. M. Dunagan: Methods for Measuring the Passage of Water Through Concrete, Proc. ASTM 866-880

[8] F. B. Hornibrook: The effectiveness of Various Treatments and Coatings for Concrete in Reducing the Penetration of Kerosene, J.A.C.I. 41, 13-20 (September 1944).

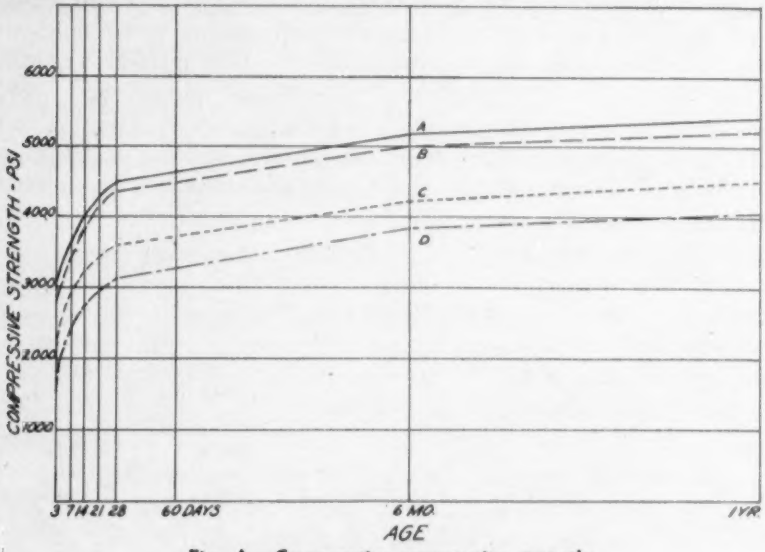

Fig. A—Comparative compressive strengths

A—5.5 sks. per cu. yd. Dispersing agent added
B—4.75 sks. per cu. yd. Dispersing agent added
C—5.5 sks. per cu. yd. No dispersing agent
D—4.75 sks. per cu. yd. No dispersing agent

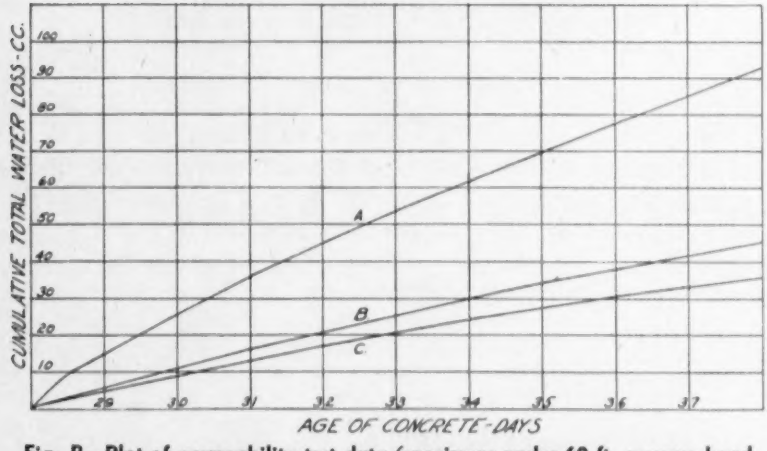

Fig. B—Plot of permeability test data (specimens under 60 ft. pressure head of water 24 hr. per day for each day shown)

A—Series "A". 6 sks. per cu. yd. No dispersing agent
B—Series "D". 5 sks. per cu. yd. Dispersing agent added
C—Series "E". 6 sks. per cu. yd. Dispersing agent added

study two agents were used which appear to correspond to Agents A and C. The permeability results are shown in Table D.

A report[9] of some tests of watertightness is shown in Fig. B.

Quite contrary to the author's reasoning, dispersion or "reduction of interparticle attraction" appears to reduce permeability. This is entirely logical and quite to be expected if the author's description of flocculation "in settling from dilute suspension the particle-flocs, in making contact with the sediment, tend to form arches or bridges, enclosing relatively large spaces which contribute to the bulkiness of the sediment", is accepted. While this is the author's description of the sediment from a dilute suspension, it is reasonable to suppose that a similar, though more condensed, structure would exist in a concentrated suspension. If so, channelled bleeding and higher permeability would be expected in the flocculated paste, not in the dispersed paste (or more compact paste with lower "interparticle attraction").

Decreased Plasticity on Standing or in Transit—A dispersing agent maintains concrete mixes in transit or on standing in a more plastic state and reduces segregation. This is a phenomenon clearly shown by field experience with concrete mixes and is especially evident and important in the ready-mix and transit mix fields. It is hardly a question that can be readily resolved by laboratory data but years of experience in the field have amply demonstrated the beneficial effects of one dispersing agent in these respects. Reference is made to one report of tests[10] and a published article[5] from among other experimental evidence on this point.

b. Agent B

1. The increase in air content was accompanied by a decrease in water content. But since the decrease in water was not as great as the increase in air, the paste content also increased in direct proportion to the increase in air content.

2. It follows that when the water content is not reduced, the increase in slump caused by entraining air in a given mix is due to increase in paste content and not to softening of the paste.

3. The data show conclusively that entrained air has a stiffening effect even though it is more fluid (i. e., it has a lower viscosity) than the water it displaces.

4. So far as workability and bleeding characteristics are concerned the effect of entrained air may be regarded as highly beneficial.

5. Entrained air reduces strength but greatly increases resistance to frost action.

These conclusions regarding the behavior of entrained air in concrete are supported in part by the experimental evidence presented, as well as much other evidence in the literature. The only phenomenon produced by this agent is air entrainment and cement dispersion is not involved. It should be noted that the author has confined himself

[9] Report of Tests, Rush Engineering Company, Chicago, Illinois, December 21, 1938.
[10] Report of Tests, Toledo Testing Laboratories, Toledo, O., April 25, 1945.

to one mechanism of air entrainment and one type of air entraining agent, namely, a surface active chemical compound which drastically reduces the surface tension of water, i. e., a wetting or foaming agent. There are other mechanisms for bringing about air entrainment, for example, the use of aluminum or hydrogen-peroxide in the mix (strictly, of course, this is not air entrainment but entrainment of hydrogen or oxygen, but it serves the same purpose). Cement dispersing agents, that is chemical compounds, which are preferentially absorbed by cement and impart an electrostatic charge to the particles, at least in dilute suspensions, also effect a certain degree of air entrainment. These reagents are not wetting or foaming agents and have only a small effect in reducing the surface tension of water. Obviously the mechanisms of air entrainment differ in each of these three cases. It is quite probable that there are other means and mechanisms of entraining air in concrete, such as, possibly, the use of protective colloids which neither affect the surface tension of water nor are preferentially adsorbed by cement. In considering the author's treatment of Agent C it should be borne in mind that the mechanism whereby Agent C entrains air is probably not that which he has described under Agent B.

c. Agent C

1. Agent C (air entraining agent and dispersing agent) reduced the water requirements more than did Agent B (air entraining agent only).

2. It is clear that the net effect was a stiffening of the paste.

3. The paste content required for a given slump increased in direct proportion to the increased air content.

4. Whatever undesirable effects on plasticity and bleeding characteristics the reduction in interparticle attraction might have, they are off-set by the entrained air

5. On the whole, it appears that *with respect to the properties of fresh concrete,* agents like B and C are beneficial, C being somewhat more so.

6. Whether or not the properties of the *hardeued* concrete are benefitted equally, or at all, by agents of this type, is a question that cannot be dealt with in general terms.

7. With respect to durability, entrained air is decidedly beneficial, and therefore the use of agents like B and C can be recommended when special protection against frost action is necessary.

8. It is difficult to see how either dispersion, per se, or a weakening of the forces of flocculation in the absence of air entrainment, could have much influence on frost resistance.

Since this agent is taken as typical of those which disperse cement and also entrain some air, the author's conclusions may be considered in the light of the behavior of a dispersing agent with which the writer has worked.

The first conclusion is entirely correct and consistent with extensive data secured by the writer on water reduction with a dispersing agent.

It is by no means clear that the net effect is a stiffening of the paste. On the contrary numerous data which the writer has accumulated show that the sum of the absolute volumes of the cement, water, and air in the mix with a dispersing agent compared with the corresponding plain mix of the same slump is less, that is, there is less" paste" in the dispersing agent mix. According to the author's line of reasoning, this proves that the paste is more fluid. Some typical data on this subject taken at random from concrete mixes prepared in the investigation of other phenomena are shown in Tables E and F.

TABLE E—CEMENT FACTOR —LB. PER CU. YD. 495

Absolute volume per cu. yd. in cu. ft.	No addition	Dispersing agent
Water	4.46	3.82
Air	0.38 (1.4 percent)	0.81 (3.0 percent)
Water plus air	4.84	4.63
Cement	2.52	2.52
Water plus air plus cement	7.36	7.15

TABLE F—CEMENT FACTOR—LB. PER CU. YD. 510

Absolute volume per cu. yd. in cu. ft.	No addition	Dispersing agent
Water	4.82	4.28
Air	0.27 (1 percent)	0.76 (2.8 percent)
Water plus air	5.09	5.04
Cement	2.59	2.59
Water plus air plus cement	7.68	7.63

Some other data taken from a series of tests made in an independent laboratory are shown in Table G.

TABLE G

Cement factor sks./cu. yd.	Slump in.	Air content —percent	Vol. cement + water + air cu. ft./yd.	Addition
5.48*	4¾	1.7	7.76	None
4.45**	4½	2.6	7.19	Dispersing agent
5.37**	5	4.6	7.72	Dispersing agent

*Average of eight mixes. **Average of three mixes.

The data immediately preceding show that, although it may be true, and probably is, that an increase in air content necessitates an increase in paste content for a given slump due to the stiffening effect of air, with one cement dispersing agent the net effect of reduction in interparticle attraction and air entrainment is to require the same or smaller volume of cement paste compared with similar mixes without the dispersing agent for a given slump. This is directly contrary to the inference to be drawn from the author's Fig. 6. This reduction of the total amount of paste required with a dispersing agent holds over the normal range of concrete mixes and the range of air content usually secured with this dispersing agent, which is fairly constant. That, as the air content was increased over 5 percent, the paste requirement might increase is entirely possible, but such air contents, as those of the upper limits of Fig. 6 which seem to reach over 10 percent, would not be secured with the dispersing agent except in some very abnormal mix. In other words, if the air content with the dispersing agent in question is kept between 3.0 percent and 4.0 percent or at most between 2.5 percent and 4.5 percent as would automatically be the case in normal mixes, the net effect is a reduction in paste volume for a given slump and by the author's line of reasoning a softening of the paste.

It is not apparent that dispersion or "reduction of interparticle attraction" has undesirable effects on plasticity and bleeding. On the contrary, data previously given show that a dispersing agent increases the effect of entrained air in reducing bleeding. The author himself shows that cement dispersion increases the beneficial effect on plasticity when he says that: "Agent C reduced the water requirement more than did Agent B".

The fifth conclusion requires no comment unless it is to point out that in reaching it the author has contradicted his whole preceding argument. After arguing that dispersion is undesirable he concludes that air entrainment is desirable but that air entrainment and dispersion are more desirable.

The sixth conclusion is non-committal.

It is well-established that entrained air, provided no other deleterious effects are introduced, improves durability. Again, after condemning cement dispersion, the author recommends Agent C which includes a dispersing agent in preference to Agent B, which is merely an air-entraining agent without a dispersing agent.

It is not at all difficult to see how dispersion, without air entrainment, can influence frost resistance. McMillan[11] has shown that

[11] F. R. McMillan: Basic Principles of Concrete Making, McGraw-Hill Book Co., Inc. 1929, pp. 38-41.

resistance to freezing and thawing (frost action) increases as the water-cement ratio is lowered. Many data of the writer as well as the data given in the paper show that a dispersing agent permits a lower water-cement ratio for a given slump than is permitted by air entrainment alone. Hence cement dispersion, among its other effects, should give greater durability than air entrainment alone. Results of recent experiments on concrete mixes with equal air contents and cement factors indicate that for a mix in which the air content is obtained through a cement dispersing agent the durability is greater than that of a mix in which it is obtained by means of an air entraining agent without dispersing action.

The foregoing data on a cement dispersing agent which also entrains some air, and other available data, referred to in part, as well as the author's data, indicate that, regardless of the mechanism ascribed to it, such a material will produce beneficial effects in concrete which cannot be secured with air entrainment alone.

By FRED M. ERNSBERGER and WESLEY G. FRANCE*

T. C. Powers has opened the discussion of a question of fundamental importance in the theory of portland cement addition agents. It has long been assumed, that the results obtained by the use of certain addition agents are due to the dispersing action of the additive. The above author takes the position that complete or partial dispersion, *if not complicated by other effects*, has a detrimental effect on the workability and bleeding characteristics of a cement paste. It should be noted that this proposition does not question the desirability and practical usefulness of commercial additives alleged to be operating on the dispersion principle: in fact, the paper concludes with an endorsement of "B" and "C" type "dispersing" agents; however, it does attack the value of portland cement dispersion in principle. The Powers proposition is therfore an academic one until an additive can be laboratory- and field-tested which has a dispersing action as defined in this paper, and no other effect. It follows that it would be irrelevant to quote data on the performance of any commercial additive as proof that dispersion has a favorable effect on the properties of concrete until such an additive is proved to have a true and uncomplicated dispersing action.

In support of his proposition the author has brought a wide variety of evidence, both direct and indirect, the chief of which is the observation that any reduction of inter-particle attraction in a paste increases fluidity. This will probably be rather generally conceded; it can be measured as an increased slump. It is possible that this increase in

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fluidity leads to increased bleeding and poor workability, but no proof is offered, beyond a faulty analogy to lean mixes, that these effects cannot be nullified by reducing the water content of the mix; the reduction in water-cement ratio remaining as a net benefit of dispersion.

The suggestion is made that portland cement particles are flocculated by hydroxyl ions. This seems unlikely in view of the fact that portland cement particles strongly adsorb the anion of calcium lignosulfanate and are thereby dispersed rather than flocculated.*

It is not wise to dismiss as insignificant the role of Brownian motion in a dispersed cement paste. While it is undoubtedly true that the bulk of a cement on a weight basis consists of particles too large for Brownian motion, there is no question that particles small enough (4 microns and below) do exist in portland cement in large numbers. Anyone equipped with a microscope can satisfy himself of this by observing a dilute suspension of cement containing a good commercial cement-dispersing agent. Because of their enormous surface, these particles naturally will have a disproportionate influence in determining the properties of fresh and of hardened cement mixtures, if they are free to make their influence felt. In an undispersed paste they necessarily move and hydrate as parts of the large particles to which they are attached by forces of flocculation. It remains to be proved that any of the commercial dispersing agents are able to free these small particles so completely that they act independently within the framework of large cement particles, but the hypothetical dispersing agent of Powers' definition would necessarily do so.

By HOWARD R. STALEY†

The factors discussed by Mr. Powers are ones that need more basic consideration than has been given in most published discussions of the specific subject. To discuss fully all the factors involved would require a great deal of space, but it is believed that the subject is of sufficient importance to warrant an even more complete discussion than can be given here.

Wetting

The discussion of wetting can be extended to consider adsorption in general, either from a physical or chemical stand-point. Wetting of a surface is probably the simplest example of the phenomenon of adsorption; and the tenacious retention of a liquid on a solid is known specifically as lyosorption. It is probable that both physical and chemical adsorption occur initially in the case of portland cement and water.

Possibly the simplest means for an engineer to consider adsorption is upon the basis of energy. It should not be difficult to conceive that

*Ernsberger and France, *Ind. Eng. Chem.* V. 37, p. 598 (1945)
†Mass. Inst. of Technology, Cambridge, Mass.

there is a certain amount of free energy or "unbalanced potential" connected with any surface and that the amount of free energy is a function of the state of subdivision of the matter. This free energy is the result of non-equilibrium conditions caused by imperfections, unbalance or strain in the lattice structure of the crystal, adsorbed ions, chemical potential or affinity, or conditions of environment; it may be either electrical or chemical in character.

There is a tendency for any system to gravitate towards a condition of equilibrium or to a state which is consistent with the lowest level of free energy. Thus, if a liquid is brought into contact with a solid and the force of adhesion between the solid and the liquid is higher than the force of cohesion in the liquid, the liquid will be adsorbed and the surface will be wetted; a condition of equilibrium has been established which is consistent with the forces and environment involved and the free energy level of the system is lowered thereby. If the conditions at the interface are modified by the addition of an agent (i.e. the surface or interfacial tension reduced) wetting may be facilitated. It should be emphasized that the modification of the adhesion forces is an interfacial phenomenon primarily. Substances that act to reduce interfacial tension are composed of molecules that have functional groups similar in character to the liquid and other groups similar in character to the solid, and these molecules orient themselves in the interface in such a way that compatible groups are presented to both liquid and solid. If the liquid is polar, as water, the polar end of the molecules will tend to be dragged into the liquid and the non-polar end will tend to be thrown out due to their dissimilarity with the liquid. With non-polar solvents, the molecules will orient themselves with the non-polar end in the liquid. In the case we are considering, where water is being used, the former of the above two conditions holds. We can say then that the degree of adsorption will depend upon the type of surface of the solid and the character of the liquid, bearing in mind that the interfacial conditions between the two can be modified markedly by other substances.

As to the wetting of normal portland cement with water, we have no assurance that the *whole* surface is wetted to the same degree, though the theory would predict that all of the surface is wetted. In this connection it might be well to point out that surface areas, as commonly determined by the Wagner Turbidimeter or the permeability method, are merely relative values and do not necessarily represent the true or even effective surface area. For example, surface area values obtained by the permeability method average about 20 per cent higher than those obtained by the Wagner method, and surface areas obtained by the nitrogen adsorption method are several times greater than for the Wagner method. A typical portland cement with a surface area of 2000

sq. cm. per gram, as measured by the Wagner method, showed a surface area of 9000 sq. cm. per gram as measured by the nitrogen adsorption method. The water molecule is approximately the same size as the nitrogen molecule in one dimension and water adsorbed initially should cover approximately the same surface area as that determined by nitrogen adsorption *provided* the interfacial conditions are the same for the gas and the liquid, which they are not. Thus, there probably is an "effective" surface area that is wetted and a different "effective" surface area that is operative in regulating bleeding, the difference between these two "effective" areas being wetting in pores where relatively stagnant conditions exist, such wetted area not being available to help in regulating bleeding. It must be considered also that the surface area changes as hydration products are produced and these "effective" surface areas change with hydration. It is probable that none of the methods used for measurement of surface area yield a value for the "effective" area, but it is believed that the area determined by nitrogen adsorption is nearer to this than values obtained by other methods.

Dispersion

We believe that the fundamental meaning of dispersion cannot and should not be avoided. If it becomes necessary to define some intangible quality, a term that has a strict definition should not be used for this purpose as such usage only adds to the confusion at a later time. The need for assigning a new meaning, which will alter by popular usage the strict definition of a term, is not apparent. There can be varying degrees of dispersion; it may be partial or complete, such degree being a measurable quantity, whatever the cause or effect.

Perhaps two definitions will be helpful along this line; a dispersing agent is a material which decreases the degree of flocculation of the discontinuous phase without lowering the surface tension of the continuous phase; a wetting agent is an agent which brings the discontinuous and continuous phase into more intimate contact by lowering the surface tension of the continuous phase.

Aggregates or flocs can be effectively broken up by dispersing agents for the forces holding the flocs together are relatively weak, even though persistent. Agglomerates cannot be broken up by a dispersing agent, for they consist of particles more or less cemented together.

Since we are concerned in this discussion with suspensions of solids in liquids, we can restrict our discussion to a simple two-phase system of this type. As pointed out by Mr. Powers, dispersion is the opposite of flocculation. It is implied then that the particles in the suspension, due to whatever cause, are present as discrete, individual units, in a suspension which is completely dispersed; they are separated by some finite distance and tend to remain as discrete particles, in suspension

or during sedimentation, as long as they are in a dispersed state. The structure built up in the sediment of dispersed particles will be granular in character, similar to the sediment built up by sand grains in falling out of suspension. The porosity of the sediment will be low as the particles tend to pack more closely on sedimentation similar to inundated sand. On the other hand the structure built up by sedimentation of flocced materials will be of a looser, honeycombed or flocculent type. Certainly the latter will bleed less but the porosity of the sediment will also be higher.

Flocculation in Pastes

Particles of normal cement form into weakly held flocs in suspension. It is indicated, however, that the flocs are more prevalent and are more tightly held together after the cement has been wetted for some time as the increase in workability is marked and this factor is a direct function of flocculation.

It is likely that the calcium ions are responsible for the flocculation but it should be remembered that there are other ions in solution, especially from the alkali metal (Na and K) which may also function in modifying the characteristics of the paste. It is believed that the hydroxyl ions would aid dispersion, also Na^+ would tend to hydrate the material more, but Ca^{++} would tend to flocculate it by a process of dehydration. K^+ acts to a much smaller degree as a hydrating ion than does Na^+.

The fact that we are dealing with suspensions of particles that are unstable in physical and chemical character due to the wetting and hydration complicates the problem of analysis. An increase in concentration of a solute, or the solution of a new compound such as may occur as the cement hydrates, affects the properties of the suspension to a marked degree. It should be remembered that the effect of an added substance which would cause either dispersion or flocculation initially might be rapidly overcome by the presence of the hydration products of the portland cement a short time later. For example, it has been noted that the addition of a flocculating agent to a suspension that has been dispersed by the use of dispersing agent in some instances causes a syneresis effect resulting in the formation of very tight flocs from which water is almost excluded.

General

Regarding the sedimentation volumes of cement in alcohol-water solutions, an investigation of the solubility of lime hydrate and the alkalies in alcohol-water mixtures should help to throw some light on the effective flocculating agent or agents in cement-water suspensions but there is another influence at work, i.e. the dehydrating effect of the

alcohol; this dehydrating effect is not present with toluene although the solubility of the flocculating agents may be changed by this substance. Such determinations should help to eliminate the necessity for hypothesis. No doubt, experiments of the sort illustrated in Fig. 1, if carried out with different concentrations of cement would show a somewhat different curve.

We are not advocates of dispersion, because we are too familiar with the beneficial effects of flocculation regarding workability and the reduction in bleeding, but it can be imagined that the reduction in mixing water to produce placeable mixes would be great enough, with complete dispersion of the cement in certain mixes, so that bleeding would be reduced somewhat. We do not believe that the above statement would hold true for all mixes, as bleeding is a function of many factors, chiefly water and cement content, the degree of dispersion or flocculation of the cement and the gradation and shape of the aggregate. It is certain that dispersion does increase the bleeding in cement pastes but we are not certain that data obtained on pastes can be directly compared with concrete mixes any more than data on dilute suspensions can be compared with measurements on pastes.

We believe that the system used for comparison in the discussion of sedimentation rate, etc., i.e. silica-water + $Ca(OH)_2$ is much too simple a system to use for this purpose. Cement-water suspensions are much more complicated because of the presence of other solubles and because of the fact that the cement grains are not constant in character as is the silica.

In the discussion on plasticity on page 129, Mr. Powers considers the characteristics of the paste as being representative of concrete mixes that contain granular aggregate materials. This comparison may or may not be true. It can be seen that the sedimenting characteristics of the paste in concrete should be much different than in plain cement pastes because of the hindrance to settling imparted by the granular materials. As a matter of fact, the concrete contains a large enough concentration of solid particles so that it is doubtful if any sedimentation occurs in the true sense of the word, but merely compression or consolidation takes place, similar to that in the compression zone in a sedimentation process as described in Perry's Handbook,* for example, and therefore stratification as visualized would not be possible.

We consider that wetted portland cement grains do not have the inherent properties that contribute to workability; that these properties are only acquired after hydration has started. It might be considered that for a very small time the cement particles would behave

*John H. Perry, Chemical Engineers Handbook, p. 1619.

approximately the same as an equal amount of silica of the same state of subdivision. The properties that contribute most to workability are not acquired until hydration progresses, and within limits; the further the hydration progresses, the more workable the mass becomes. This is in accord with observed facts. Since workability is a function of the degree of flocculation and wetting, the tendency for normal cement particles to flocculate must increase with time. Initial dispersion might be beneficial in that the cement particles could be more uniformly distributed throughout the mass, then with the increasing tendency to flocculate, that would exist under any circumstances, and with the increased amount of hydration products more equally distributed throughout the concrete, workability might be enhanced. No one can argue that a single particle cannot be wetted more thoroughly and more quickly than a clump or aggregate of particles and the amount of hydration products might be increased by this means as well as resulting in the latter being more uniformly distributed. As far as we know it has not been definitely established whether the hydration products are fibrillar or corpuscular in character but, in any event, it is probable that these products are more colloidal in character than the original cement grains. If these products can be dispersed so that the unhydrated particles can be wetted easier and hydration can continue at a more rapid rate, dispersion would be beneficial. Supposedly, the photographs we have seen of dispersed cement grains were taken within a short time after wetting, but this is not definitely known. The time factor must be given consideration as the behavior of the unhydrated cement grains cannot be taken as being typical of the hydration products.

In the discussion on page 131 regarding the addition of oleic acid to a cement suspension in kerosene we fail to see "that varying the concentration of a dispersing agent will change the consistency, cohesiveness and bleeding characteristics of a paste even though actual dispersion is not produced." Presumably the action of the oleic acid in this instance is to cause dispersion though other phenomena might occur simultaneously. The fact that the suspension becomes more liquid-like in character, is an indication of dispersion. In any event, it is questionable if cement in kerosene is comparable to cement in water.

With regard to the use of agent "A" some dispersion must occur to result in a softening of the paste. Here it is assumed that "softening" means becoming more liquid-like. It is possible that if some agent were present to accelerate the rate of hydration of the cement as well as a dispersing agent, dispersion could be accomplished and still have no noticeable reduction in volume of the paste. Since many of the so-called dispersing agents for cement contain two or three substances,

one of which is an accelerator, the above explanation may be consistent with the facts of the case. To conclude that workability was enhanced by the addition of this substance would be to conclude that slump is a measure of workability.

In the discussion of other dispersing agents it was stated that all except agent "A" caused air entrainment. This brings up the question as to whether the air content was actually measured or whether the gravimetric determination was used for this measurement. We have found the gravimetric method to be somewhat unreliable for exact measurements of this quantity. Regarding the entrainment of air one would expect a stiffening effect due to this since there is more surface present. It is somewhat analagous to replacing a low specific surface cement with one of high specific surface with the consequent stiffening effect.

To conclude this discussion, the scope of the principles and subjects covered in Mr. Powers' paper is too great to cover completely, though a thorough understanding of the subject matter is necessary for an understanding of the phenomena that occur in the handling of portland cement mixes. We are inclined to believe that many of the statements made by Mr. Powers regarding interparticle attraction are questionable but on the other hand they may be substantiated by more precise measurements at some later time. Until more precise measurements are made on some of the factors discussed, the necessity for hypothesis will continue to exist.

By C. A. G. WEYMOUTH*

In entering the discussion of the controversial question asked by Mr. Powers, I find myself in the exceptional position of being in agreement with the fundamental arguments of both sides. Not only have I had satisfactory experience with cement dispersion for the past decade in architectural concrete and see many advantages in its use, but I also agree with Mr. Powers that flocculated cement paste is essential to plasticity in concrete. In fact I am on record in this latter regard in my outline of a theory of the mechanism whereby mixtures of coarse and fine aggregates, cement and water become plasticized.† If both dispersion and flocculation are good things for concrete, how can both be simultaneously possible? And if this is so, will a recognition of this fact afford a common meeting ground for the various interests involved?

The first question is answered in the affirmative by some simple tests properly explained; the phenomenon must have been observed many times without its significance becoming apparent.

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†See my discussion of Powers' reference 7a.

The tests make use of the standard procedure of soil analysis by the hydrometer method using a 1000 ml. glass cylinder. Cement becomes completely flocculated when tested in distilled water, or in a column of plain kerosene; the suspension settles, fine with coarse particles, without separation of sizes leaving a column of clear liquid above. However, when a dispersing agent is added to a sample of cement in the dispersing cup, the coarsest cement particles quickly settle through the column giving a black sediment, which upon examination will be found to be very compact. This is followed by a gray sediment of finer grains, likewise dispersed and compact. Up to this point the actions in a water and a kerosene column are in agreement. The finer suspension in kerosene continues to give a stratified sediment until settling is completed; the finer suspension in the water column above the sediment, however, changes to a flocculated condition. This is most noticeable with the commercial agent, Pozzolith, which carries calcium chloride, when used in the water column in the least amount to give the dispersed sediment. From this point of change the suspension settles rather rapidly (compared to the final settlement in the kerosene column) and without stratification of sizes. The dispersed sediments are so compact that there is difficulty in breaking their formations loose by shaking, while the flocculated sediment can be thrown into the water column again with little effort. By observing the times of settlement, the dispersed grains appear to be coarser than from 10 to 20 microns. These actions can be checked by a number of interesting variations in the tests.

It seems possible, therefore, to have both dispersed and flocculated grains in the same cement paste. This word "dispersed" is here taken to imply a breaking or elimination of the bonds of attraction of other adjacent elements to the grain in question. This is not in accord with the definition given on page 123 of the paper under which the grains are not to be termed "dispersed" unless this breaking of the bonds has no appreciable effect on the physical properties of the paste. The elimination of these bonds from the coarse grains in the paste does, however, have very beneficial effects on the physical characteristics of the paste.

Assuming that commercial dispersing agents do break the interparticle bonds to the coarse cement particles in concrete, and assuming Mr. Powers' picture of the continuous network of flocculated particles in undispersed paste, a reasonable explanation can be stated for the increased slumps in concrete due to this type of dispersion. Without dispersion the large cement grains are held apart from each other by chains of fine flocs which limit their freedom of interparticle movement and stiffen the paste somewhat like internal friction. The attachment of a fine floc to a coarse particle is tighter than the attraction between

fine flocs to each other. Dispersion frees the coarse grains, together with the water associated with them, from this particle interference and from the restraining influence of the flocculated network still remaining in the paste. This water now lubricates the mix more effectively and gives a wetter consistency. In construction concretes, dispersing agents increase the slumps for a given water-cement ratio, and make possible the reduction of the water content for a fixed slump by from 2½ to 3 gal. per yd. Dispersing the coarse cement grains improves concrete by giving it increased density, greater plasticity, less shrinkage and greater strength because of the lower water-cement ratio.

The tests cited indicate a relationship between the size of the cement particle and the density of the coating of hydrates: that this density increases with the mass of the grains, and that active water is thus prevented from reaching unhydrated surfaces on coarse cement grains under the coating to continue chemical reactions. The tests also indicate that an adsorbed dispersing agent on a coarse grain remains undisturbed, whereas its effect on fine cement grains is nullified after a short time. I find no generalization in text books on the physics of surfaces relating the thickness of an adsorbed water film to the mass of the particle, but that this relation is true is strongly indicated by studies on soil compaction to various critical conditions which show that much thicker water films are adsorbed on large grains than on those with very small diameters.* Conversely, adsorption of the products of hydration and possibly the solution of a dispersing agent is far less dense on individual fine grains. Hydration thus continues for a longer time on fine grains and in accordance with the law that free surface energy increases rapidly as the radius of the particles diminishes. Loosely held hydrates have been reported being pushed off into the spaces between the grains to form a continuous gel as the reactions proceed. This action cancels the negative charges on the fine grains put there by the original dispersion after which the fine particles of cement become flocculated.

By M. SPINDEL†

The paper under discussion is another step forward to solving one of the most complicated problems by an author who since 1932 has already contributed so much to this question in one way or another.

It was a bit of a disappointment that the author based all the results regarding the effect of some typical admixtures, on the slump test alone, without giving figures as to how the entrained air and the more or less reduced water-cement ratio influenced also the flowability and the workability. If it was correct (and to a certain degree it really was)

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†Consulting Engineer, London, Eng.

that reducing the intensity of interparticle attraction did not mean improving the workability, then the slump test alone was not at all suitable to demonstrate the effect of the agents A, B and C, although the tables and figures showing these results were of great interest.

Does the author mean that the slump test alone was really sufficient to demonstrate the effect of three typical agents A, B and C on concrete, and if not, which tests would he consider more suitable for this purpose?

Why did the author in his tables and figures not show also the effect of the typical admixtures A, B and C on the strengths of concrete as it was being done in most of the investigations regarding the effect of the water-cement ratio and also regarding the effect of the void-cement ratio by authors of the Symposium "Concretes containing Air-entraining Agents" and also in the paper "Tests of Concretes containing Air-entraining Portland Cements or Air-entraining Materials added to Batch at Mixer", by H. F. Gonnerman*? Of course it is true that the loss of strengths caused by the agents B and C and the gain in strength caused by agent A could be calculated from the tables and figures of the paper as to well known formulae, but as a rule it is not being done unless there is indication in the paper that it was necessary to do so and above all that the percentage of entrained air had to be limited to a few percent only.

I cannot agree to the author's statements that "portland cement is highly hydrophilic" and that, therefore, "the wetting of portland cement by water could hardly be improved by a wetting agent". The author's chief argument that the surfaces of cement grains react with water chemically does not suffice to prove the above statement, because even the cement grains which acquired a special "water-repellent" coating have to react and do react with water nearly in the same way. The difference is that the latter want some more water for normal consistency of the neat cement paste and that much more work is necessary for thoroughly mixing the paste if the water-cement ratio is not sufficiently increased. What matters is the time and the amount of energy needed to mix the cement grains thoroughly with the least amount of water possible for the purpose in question. This amount of energy and the minimum of mixing water will depend upon the means used for the best distribution of the cement grains in water which above all should not be handicapped by pockets of water which might be caused by too rapid and too intense flocculations.

The author certainly referred to commercial portland cements when he stated that even when spontaneous dispersion is not possible, dispersion can be effected mechanically, what is sometimes being done indeed on the job by using special expensive apparatus to separate the

*A.C.I. Journal June 1944, Proceedings V. 40, p. 477.

cement particles by violent stirring. There is no reason why the same effect should not be achieved rather by suitable admixtures if it can be done. The author was right that "the forces of interparticle attraction" must not be cancelled or reduced below the minimum necessary for a plastic paste. Unfortunately that disastrous effect is often being caused by an excess of water in order to obtain the necessary flowability and workability and also by an excess of entrained air. Therefore, in my opinion, chemicals which prevent such an excess of water are helpful to avoid great trouble from reducing the forces of interparticle attraction.

Apart from the well-known usual methods of testing hydrophobic and hydrophilic materials I used a very simple test of my own to show even without a magnifying glass the differences between the fine grains of highly hydrophilic materials, such as, gypsum or loam, and of highly hydrophobic materials, such as stearates and of a series of grains which are between these extremes, to which portland cement grains belong. One can see this difference also by scattering some small particles on the surface of a drop of water. The more or less hydrophobic grains, including those of portland cement, will remain on the surface whilst the more or less hydrophilic grains of gypsum, loam or even quartz will sink into the drop of water.

As a matter of fact we cannot say that a material is either hydrophobic or hydrophylic since there are various degrees between both depending not only on the chemical composition and the shape of the surface but, above all on the percentage of the area to which air is adsorped. It might be that the author made his tests and statements under special conditions. Perhaps to avoid theoretical misunderstandings we should use the word "distribution" instead of "dispersion" and then we shall rather agree that special means for better "distribution" of the cement grains in a minimum of water appear to be very useful, if chosen and used in the right way.

Although there are some more views which have still to be discussed, everybody will agree that we have to be very thankful to the author for his latest contribution to solving these problems.

By EMIL SCHMID*

The action of a solution of dispersing agents in water on cement is not necessarily the same as it would be on inert powders. Most dispersing agents react chemically with cement or act physically in respects other than dispersion so that the effects of dispersion are distorted or overshadowed. Additional effects of these agents may be either one or a combination of the following:

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1) Increase or decrease of surface tension, resulting in slower or more rapid wetting of the cement and possibly also in reduced or increased penetration of water into the cement grains during mixing and before cement particles become coated with hydrates.

2) Formation of complex soluble salts resulting in dissolution of part of the initially formed hydrates.

3) Acceleration or retardation of the hardening rate.

4) Reduced or increased air entrainment due to increased or decreased surface tension.

In addition thereto commercial preparations may contain accelerating, porefilling, air entraining, or other agents which may or may not interfere with the effects of dispersion.

Dispersion has been credited directly and inferentially with many effects, which are known to be caused by other physical or by chemical action. One of the merits of Mr. Powers' paper is to expose and correct these misleading claims. Experience and data presented by Mr. Powers seem to indicate that complete dispersion is undesirable from the point of workability and bleeding (quantity of water released, regardless of rate or type of bleeding).

Agents have been tested by us which, when used in sufficient quantities, disperse cement so completely that concrete containing 7 bags cement per cu. yd. appears like a wet mixture of fine powder, sand, and aggregates and has no plastic properties at all. Such concrete, however, shows an increase in compressive strength of over 60 percent at 28 and 90 days. Although dispersing agents may have an undesirable influence on one property of concrete, they may improve others.

Concrete which cannot be placed is almost as worthless as Kennedy's "concrete" which can be placed, even though it contains no cement. This proves only that the question cannot be put "Should cement be dispersed?" or "Should air be entrained in concrete?" as it all depends on how much, for which purpose and which type of concrete is involved.

Dispersion therefore cannot be discussed by describing its influence on workability and bleeding alone. Increased fattiness or cohesiveness of the plastic concrete may facilitate placing slightly but has no relation to the service a structure will render. Although the paper states that actual merits will not be discussed attention is drawn to the fact that air entraining compounds or dispersing agents having air entraining effects show favorable influences on frost resistance. It is therefore only fair to mention improvements to be obtained by other, not necessarily air entraining, agents—the more so as some of the conclusions reached by Mr. Powers are contradicted by test results.

In view of the occurrence of other effects dispersing agents produce in concrete it is quite possible that dispersion alone would not cause all

improvements listed. However, whereas Agents B and C exert definite influences which, according to Mr. Powers, offset some of the results of dispersion, Agent A is the only one mentioned producing effects which rather enhance than impede the characteristics of dispersion. Being manufacturers of a concrete densifier equal to Agent A, we have considerable experience in regard to its influence on various types of concrete.

A dispersing agent only entrains air; if in alkaline solution, it lowers the surface tension of water. Agent A increases the surface tension slightly and the air content is therefore unchanged or reduced, resulting in rather more than less fluidity of the paste.

The article states that cement grains become coated with hydrates. These hydrates are in the form of gels, increase the volume of the grains, and absorb water molecules. It can be assumed that such enlarged particles of average lower specific gravity have less tendency for settlement than the original cement grains. Such hydrates combine with a solution of Agent A to form complex soluble salts. Less gel therefor surrounds the cement grains.

It should be expected that the effects of dispersion in regard to workability, sedimentation, segregation and bleeding would tend to be emphasized rather than hindered by the secondary effects of Agent A (increase of surface tension and decrease of gel formation).

The results mentioned below have been obtained with Agent A and may or may not be applicable when comparing the influence of other agents on concrete.

Workability

The effect of dispersion or better deflocculation or distribution of cement in concrete on workability as produced by Agent A has been known for a long time. Its action is illustrated in Fig. C.

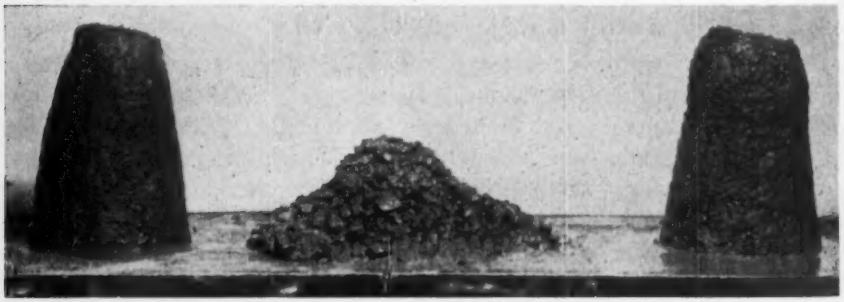

Fig. C—The effect of Agent A on slump and water-cement ratio of concrete, 5¼ bags of cement per cubic yard.
(Left) Without Agent A slump 1¾ in. Water-cement ratio: 0.54.
(Center) With Agent A slump 7 in. Water-cement ratio: 0.54.
(Right) With Agent A slump 1¾ in. Water-cement ratio: 0.44.

This photograph was depicted by Mr. Powers at the ACI convention in Buffalo, N. Y., but has been published in all our illustrated literature since 1934. Attention should be drawn to the fact that the concrete used is undersanded and leaner than recommended by us for best results. The increase of fluidity on workability is undesirable in lean or undersanded mixtures, but shows no ill effects if used with well proportioned concrete containing over 5½ bags cement per cu. yd.

Sedimentation and Segregation

In a fluid suspension particles may stratify when forming sediments. In a paste as used in concrete this danger does not exist. Even with extremely plastic and flocculated air entrained concrete continuous slow mixing is necessary (Alexander Foster* of the Warner Co. in Philadelphia during discussion of air entraining agents at the 1946 ACI convention in Buffalo, N. Y.) if long waiting is required. Concrete of partially dispersed cement as produced by Agent A, when agitated in the above manner, will not form rigid sediments either.

Segregation is reduced if the entire mass of the concrete is truly plastic and cohesive. It is also reduced if the cement is dispersed and not cohesive. This paradox can be explained as follows: Plastic cement paste is cohesive but not adhesive because the attraction of cement particles to each other (flocculation) exceeds the attraction to aggregates or steel. The film of water surrounding the hydrates prevents such adhesion further. A chute or shovel, or for that matter, reinforcing steel and aggregates, remain comparatively clean after withdrawing from concrete. This can best be observed in concrete of 3 in. to 5 in. slump.

If, however, the cohesiveness of paste is destroyed (and in case of Agent A, the formation of hydrates reduced) cement particles stick to whatever they contact, be it aggregates, steel or hardened concrete. Such concrete may therefore fall apart, but the aggregates remain covered with cement particles.

For example, it has been observed in road work that when concrete is screeded with a finishing machine, excess plain concrete is pushed forward without turning over; whereas concrete containing Agent A falls over and turns continuously as the screeding blade progresses.

To determine the degree of segregation, the deviation from the average of a large number of test results has been calculated. This deviation using plain concrete is usually over 50 percent higher than it is using controlled air-entrained concrete (plastic mixes) or using Agent A concrete (dispersed mixes). Either method therefore decreases segregation.

Another form of segregation, characterized by the formation of scum, can be caused by bleeding as discussed below.

*ACI JOURNAL, June 1946, *Proceedings* V. 42, p. 625.

Bleeding

The paper states that (a) flocculated pastes bleed slower than dispersed pastes, (b) bleeding is increased by dispersing agents and (c) during bleeding the aggregates form a static frame work, which prevents settlement of the concrete and therefore allows accumulation of water beneath aggregates or steel.

It is also stated, that accumulation of water occurs in plain concrete underneath steel and aggregates, but that the use of air entraining agents will reduce bleeding and is beneficial in eliminating water fissures. Following through with this deduction, it should be expected that air entraining agents increase flexural strength and bond to horizontal steel bars. Charles E. Wuerpel presents some interesting facts in regard to the above in his paper "Laboratory Studies of Concrete Containing Air Entraining Admixtures" (ACI JOURNAL Feb. 1946). In spite of the fact that (as indicated by the results of the compressive strength tests) the strength of the paste is not reduced when using air entraining agents, the flexural strength is not increased (see Table 5, Admixture R). Table 17 gives the test results of bond strength to horizontal steel bars. Whereas compressive strengths for the lean mixes were rather increased by the use of air entraining agent, the bond strength to steel for these mixes was considerably reduced—the opposite from what should have been expected according to Mr. Powers' theory.

Mr. Powers' deductions in regard to bleeding are contradicted by the results of the following series of tests carried out by Smith-Emery Company's laboratory. The concrete tested had the following composition:

	Series I	Series III
Mix	1:2.55:3.53	1:2.61:3.60
Agent A	none	1 lb. per bag cement
Bags per cu. yd.	5.65	5.65
w/c gal./bag	6.40	5.89
slump ± ¼ in.	4.00	4.00

The water cement ratio in Series III has been reduced so that the same workability as Series I was obtained. This is the procedure usually followed in construction work.

1) *Bleeding* 6 x 12 in. watertight steel cylinders were filled with the concretes to within ½ in. from the top and a depression made in the center. The quantity of water bled was removed at 15 min. intervals and measured. The curve obtained disproves deductions made both in regard to total quantity of bleeding and rate thereof. (See Fig. D.)

2) *Flexural strength* Tests at 28 days, third point loading, 30 in. span, average of 5 beams each, produced the following results (same composition of concrete as given above)

Series I (plain concrete) 654 psi
Series III (Agent A added) 734 psi

The increase of 12 percent in flexural strength is at least equal to the increase to be expected from the reduction of slightly over 8 percent in the water cement ratio.

3) *Adhesion to steel* (pull out tests, 0.001 in. initial slip at the free end)

Three specimens 6 x 6 x 10.5 in. were cast with the $\frac{7}{8}$ in. round transverse lug reinforcing bars placed in the center of the specimen in a vertical position.

Three specimens 6 x 10.5 x 18 in. were cast with the $\frac{7}{8}$ in. round transverse lug reinforcing bars placed in a horizontal position. The bottom bar was placed 3 in. from the bottom and the top bar was placed 3 in. from the top. After these three specimens were cured for 24 days they were sawed, making three 6 x 6 x 10.5 in. specimens with bottom cast horizontal bar and three 6 x 6 x 10.5 in. specimens with top cast horizontal bar. After sawing, curing was continued until tested at the age of 28 days.

Results were as follows: (average of 3 determinations)

Location of bars	Series I (plain concrete)		Series III (Agent A added)		percent of improvement over plain concrete
	bond, psi	percent of vertical	bond, psi	percent of vertical	
Vertical	790	100	899	100	14
Bottom horizontal	603	76	956	106	58
Top horizontal	323	41	688	76	113

This test disproves conclusion reached in the paper in regard to influence of dispersion on bond strength caused by Agent A.

4) *Channeled bleeding* Should channeled bleeding occur, it has not been proven that its effects are harmful. When water is withdrawn from fresh concrete by means of vacuum or absorptive linings, the "danger" of channel formation is much more pronounced than it is in case of dispersed pastes. As is well known "case hardened" concrete shows greatly improved qualities in regard to resistance to frost and absorption.

If channeling should exist and show ill effects, resistance to sulfates, for instance, would be reduced. The above concretes were tested in accordance with the ASTM sodium sulfate soundness test method using

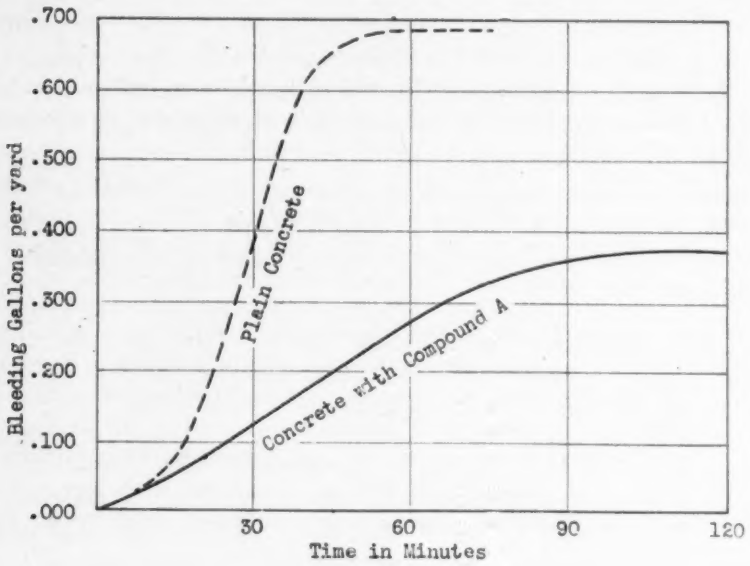

Fig. D

50 cubes for each concrete. Plain concrete specimens disintegrated after 25 cycles, whereas the concrete containing Agent A withstood 45 cycles.

5) *Sedimentation* Whether bleeding is harmful entirely depends on the effects it produces. Proven above is the beneficial influence Agent A exerts on bond to steel and soundness.

The most common objection to ordinary bleeding, however, is due to the fact that it may cause scum and laitance to be deposited on the surface of concrete. Flocculated pastes apparently are carried to the surface by rising water and air bubbles. In a dispersed state the cement suspension clings less to the escaping air and a considerable reduction of the formation of scum and laitance is observed. This reduction is greater than what could be expected from the decrease in bleeding mentioned above. Kennedy has demonstrated in his discussion on air entraining agents* that the strength and surface hardness of concrete vary considerably when lower and upper sections of specimens are tested. This difference is still more pronounced if hardness of bottom and top surfaces is determined. Results obtained with Agent A seem to indicate that stratification is reduced. They prove that formation of weak surface layers is eliminated.

Concrete slabs, 6 x 12 in. 5½ in. thick, were poured of concrete Series I and III mentioned above and finished with a steel trowel, simu-

*ACI JOURNAL, June 1946, *Proceedings* V. 42, p. 641.

lating the method used for finishing concrete floors. The specimens were cured for 28 days and dried to constant weight. The resistance to abrasion was determined by blasting steel grit on sections of the top and bottom surfaces of the specimens. Results were as follows (average of 6 determinations);

	Series I (plain)		Series III (Agent A)		percent of improvement
	loss	percent of top	loss	percent of top	
Top	16.58 gm.	100	12.84 gm.	100	29
Bottom	12.73 gm.	77	11.59 gm.	90.4	10

6) *Absorption* In the paper it is deduced, based on the presumption that larger fissures occur, that dispersion as produced by Agent A makes concrete more permeable. This statement is sufficiently contradicted by E. W. Scripture, Jr.'s discussion on this subject. Be it only added that in the test series mentioned above, 24 hour absorption of Series I (plain concrete) was 5.23 percent, whereas in Series III (Agent A added) it was only 3.57 percent.

Effects of paste content

It has been shown that improvements caused by Agent A are more pronounced than can be explained on the basis of the reduction in bleeding. The decrease in paste volume is responsible therefor. Shrinkage, strength, impermeability, density, as well as resistance to injurious solutions and freezing and thawing are dependent on the quality, quantity, and continuity of the paste in concrete. In lean mixes the quantity of the paste is not sufficient to obtain a dense matrix. Such concrete is therefore benefitted in various respects by an increase in paste content as caused by entrained air. In concretes containing 5½ or more bags cement per cu. yd. such increase in paste content is only desirable in case of not well proportioned aggregates. As richness of a mix increases, the percentage of improvement caused by air entraining agents decreases. Vice versa, effects as caused by Agent A may not show great improvements if employed in lean concrete because, although the paste is dense, it is of insufficient volume to fill all interstices between the aggregates. Strength increases, as obtained due to the possible reduction of the water-cement ratio may be present but improvements in regard to other properties listed will not be as pronounced as with richer concrete, in which a reduction in paste volume is possible without damaging the continuity of the paste.

Tests show that Agent A, when used with a 4½ bag concrete, increases the resistance to freezing and thawing only insignificantly (according to

the water-cement ratio law); whereas, if used with a 7 bag concrete, the resistance is increased over 500 percent.

As long as the quantity of the cement paste remains sufficient to fill all interstices between the aggregates, it seems apparent that the more compact and dense such paste is the higher will be the quality of the resulting concrete in regard to adhesion, hardness, strength, watertightness and resistance to injurious solutions and freezing and thawing.

The tests presented prove that beneficial results are obtained not only in case of "mixes unusually rich in cement" but with concrete of as low as 5½ bags of cement per cu. yd. Such mixes, and even richer ones are commonly used for structural concrete where improvements, outlined above, are often desirable.

The evidence submitted in this discussion is not intended to detract from the value of the experimental data presented in Mr. Powers' paper and the well founded theories developed by the author in regard to the behavior of pastes as such. It is presented to draw attention to the fact that the properties of concrete cannot be deduced from the action of pastes alone. The gradation and proportion of, as well as the quantity of cement in relation to the aggregates and the water-cement ratio influence results to such a degree that the quality and behavior of concrete cannot be predicted from an investigation of dilute suspensions or concentrated pastes.

By LOUIS R. FORBRICH*

Since the use of certain additions to concrete has become popular, particularly the dispersing agent and air-entraining types, there has been considerable confusion and misunderstanding regarding the true effects of these additions. Perhaps the main reason for the confusion pertaining to the dispersing agent type, has been that the dispersing agent itself usually affects the rate of hydration of the cement and sometimes causes air entrainment. Part of the confusion, indeed, may also be due simply to definition of terms.

From Mr. Powers' paper it appears that most of the materials tested containing dispersing agents did not cause portland cement to be dispersed in accordance with the stated definition of dispersion. That is, the agents used in the amounts recommended did not cause cement to remain dispersed after mixing, although apparently the agents did reduce inter-particle attraction to some degree. From the discussion and data presented, it would seem unlikely that portland cement could be dispersed or long remain dispersed as defined, since as soon as cement and water come into contact, or shortly thereafter, electrolytes are formed which acting as flocculating agents oppose the forces of repulsion

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resulting from the dispersing agent. Whether or not a cement could be made to remain in a dispersed state for a significant period after mixing would appear to depend to a considerable degree upon the amount of dispersing agent present.

If portland cement were an inert material there perhaps would be little question that it could be readily dispersed by a suitable dispersing agent. However, since portland cement is not inert and reacts quickly with water to form coagulating ions, it is difficult to see how it could be made to remain in a dispersed state as defined for a significant period after mixing, except perhaps for the very smallest cement particles.

Since additions containing cement dispersing agents generally permit less mixing water and produce certain desirable effects in concrete, it is of interest to know more about what the action of these additions is, or why they are called cement dispersing agents, if they do not cause the cement to remain dispersed, as defined. In this connection it may be of interest to mention very briefly the results of some tests made by the writer a few years ago to determine the effects of a dispersing agent and type of mixing on the water requirement and viscosity of cement pastes. Typical data obtained are shown in Table H and the main conclusions are summarized below:

1) By vigorous mechanical mixing (640 r.p.m.), the water required for a given consistency was 10.4 percent less than what was required for a cement paste mixed by hand (using an egg-beater).

2) When mixing was done by hand, the addition of a dispersing agent reduced the water requirement of the cement paste 12.5 percent.

3) When mixing was done mechanically, the addition of a dispersing agent reduced the water requirement by only about 5 percent.

TABLE H

Addition	Type of mixing	Mixing time min.	Water cement ratio by wt.	Stormer viscosity sec. per 50 revolutions 200 g. wt.
None	hand	5	0.48	15
None	machine	5	0.43	14
Dispersing agent	hand	5	0.42	14
Dispersing agent	machine	5	0.40	13

These tests showed that thoroughness of mixing had an important influence on the water requirement of a cement paste, as well as did the use of a dispersing agent. Since vigorous mixing and the addition of a dispersing agent effected substantial reductions in the water requirement of a cement paste, and to about the same degree, it would appear that the action of a dispersing agent may be quite similar to the action pro-

duced by vigorous mixing. Vigorous mixing tends to separate or disperse particles by overcoming or reducing the intensity of interparticle attraction. Dispersing agents weaken inter-particle attraction, thereby facilitate the separation of cement particles which with normal mixing may tend to stick together. This may well be the reason why cement dispersing agents are so named.

The action of a typical dispersing agent may be briefly and tentatively described then as being similar to that which is produced by vigorous mixing; namely, it separates or disperses particles of cement during mixing, which with ordinary mixing may tend to stick together. After mixing is completed, the cement particles become surrounded with an absorbed layer of water or hydrates,, and a flocculating condition is reached in a comparatively short time. The cement paste then would be expected to behave similar to any other flocculated cement paste except mainly for the effects resulting from the lower quantity of mixing water required.

By HENRY L. KENNEDY*

I have read Mr. Powers' paper with a great deal of interest, and I have enjoyed his able discussion of the subject.

I agree with Mr. Powers in what I believe to be his conclusion, namely, "Dispersion alone is not enough." We have known this for many years. As a matter of fact, this statement was included in our advertisement in the Technical Progress Section of the 1943 ACI JOURNAL.

Indeed, to the best of our knowledge, no material is being promoted, or has been promoted during the past twelve years, either to the cement mill or to the producer of concrete, which is a true, unadulterated dispersing agent. It would seem that the caption of the paper concerns a matter of purely academic interest, particularly since the author states repeatedly that his discussion is based entirely on a product whose predominant effect is dispersion of the cement.

Dispersed portland cement is a meaningless expression unless qualified by degree. Reduction of interparticle attraction is a quantitative expression that may be applied to partial dispersion, but the extent or degree must be known to commend or condemn, since partial dispersion may be just short of flocculation or just short of complete dispersion.

A suggested caption, in step with present-day progress in portland cement concrete technology, might be "Can the Principles of Colloid Chemistry be Utilized to Improve the Quality of Portland Cement Concrete?" This would include a study of cement dispersion alone and in combination with other effects.

*Dewey and Almy Chemical Co., Cambridge, Mass.

In the time allotted me for this discussion, I would like to point out a few items which may clarify some issues embodied in Mr. Powers' paper.

1) The *very, very* broad subject of dispersion of portland cement paste (reduction of inter-particle attraction) is discussed; not dispersing agents. I do not know how many dispersing agents Mr. Powers worked with, but I do know our own research involved hundreds of dispersing agents used in concrete tests with varying and contradictory results. As a result, we have found that dispersing agents causing true dispersion may at the same time function as wetting agents. A dispersing agent when used in larger quantities may lose its dispersing effect and become a powerful wetting agent, and wetting agents when used in larger quantities may lose their wetting effect and become powerful dispersing agents.

Hence the exact type and *quantity* of a particular agent governs its effect in portland cement concrete. This is true for either a dispersing agent, a wetting agent, or an air entraining agent.

2) Tests with dilute suspension of cement and tests on portland cement paste alone containing a dispersing agent or an air entraining agent are not necessarily a criterion for concrete performance, even when the same or similar pastes are combined with aggregate to make concrete. Incidentally, in concrete comparisons an adjustment in water content is usually made in order to work to a comparable consistency, usually measured by slump. In cement paste comparisons, differences

TABLE I—DURABILITY OF NEAT CEMENT SPECIMENS AND CONCRETE SPECIMENS CONTAINING DISPERSION AND AIR ENTRAINING AGENTS

Cement	Addition	Neat cement specimens 1½" x 1½" x 6" prisms			Concrete specimens 1½" x 1½" x 6" prisms		
		Wgt./ cu. ft.	*Cycles of freez. & thaw.	% Loss in dynam. mod.	Wgt./ cu. ft.	*Cycles of freez. & thaw.	% Loss in dynam. mod.
A.	None	139.5	53	0.0	150.0	16	48.7
A.	.03% A E agent	135.0	53	16.5	136.0	16	23.4
A.	.2% dispersion agent	138.0	53	4.3	148.5	16	44.4
B.	None	139.5	200	26.5	151.5	10	51.5
B.	.03% A E agent	137.0	200	100.0	139.0	10	17.1
B.	.2% dispersion agent	139.5	200	35.1	151.0	10	33.1

*Specimens Frozen and Thawed in 10% $CaCl_2$ Solution.

in consistency are often neglected. As should be the results of the comparisons so made!

Table I illustrates a paradox existing between air-entraining cement paste which shows poor durability, and air-entraining concrete, which shows excellent durability.

Indeed, the outstanding benefits of air entraining portland cement concretes would probably never have been realized if the concrete technologists had limited their studies to air entraining portland cement pastes alone.

3) There are two phenomena that no longer require speculation:

a) dispersing agents will lubricate or make more fluid the cement paste of a portland cement concrete. I define lubrication as a reduction in friction between the particles of cement due to reduction of interparticle attraction when dispersed. The net result is the same as adding more water. It can be easily demonstrated that true dispersing agents do not lubricate, or make more fluid, fine aggregate alone.

b) In contradistinction, an air entraining agent will stiffen the cement paste but will lubricate the fine aggregate of a portland cement concrete by the formation of a number of discreet air bubbles which produce lubrication by means of a "ball bearing" effect. The degree of lubrication will vary with the amount and type of air entraining agents used, the physical properties and mechanical analysis of the fine aggregate and the type, time and speed of mixing.

These lubricating effects can be easily demonstrated by a simple test procedure where the dispersing agent or the wetting or air entraining agent is added to the cement paste alone, and also to dampened fine aggregate alone.

Until I see more tangible evidence that stiffening of the paste contributes to the plasticity of air entraining concrete, I shall continue to believe that the action of the air entraining agent on the fine aggregate is the prime contributing factor.

I agree with the author in his description of paste made in the presence of an air entraining agent as being stiffened, but I believe his description of the "softened paste" from dispersion is an unfortunate selection of nomenclature, because softening infers softness or "punkiness" of the hardened paste, which would lessen its strength and durability. I do not believe this inference was intended.

Fig. E shows results typical of many tests conducted in our laboratories. You will note the relative durability of the cement paste made with the dispersing agent as compared with the paste made with no agent. You will also note that the stiffened air entraining paste did not stand up under the action of freezing and thawing as well as either the dispersed or the blank pastes.

Fig. E

To conclude, let me repeat in part the sixth paragraph of Mr. Powers' synopsis, which is of fundamental significance:

> "Air entrainment together with some reduction in interparticle attraction affects paste content and water requirement in the same way as air entrainment alone, but the increase in paste content is smaller *because* the reduction in water content is greater than when there is no reduction in interparticle attraction."

Note that I have changed but a single word, "*and*" to "*because*." Mr. Powers might have added that because of the reduction in water content, the paste should be of higher quality.

Mr. Powers is to be congratulated for bringing out for open discussion a most timely subject. This discussion plus additional research may lead to vastly improved concrete by utilizing dispersion in combination with other materials.

AUTHOR'S CLOSURE

The discussions of this paper indicate that both the letter and intent of the paper have generally been misunderstood. I shall therefore attempt to clarify some of the topics and if possible refocus attention on the subject dealt with in the original paper.

Let us note first that in normal cement pastes the cement particles have an attraction for each other. When they collide, they tend to stick together. This constitutes a state of flocculation. In dilute suspensions this state is manifested by visible individual clusters of grains called flocs. In cement pastes in the practical range of concrete mixes it is not manifested by individual flocs because all the grains are linked together into a continuous network, actually a single floc. In such pastes

a state of flocculation is manifested by the nature of the bleeding characteristics and flow properties.

The question under discussion is this: Is this normal state of flocculation undesirable? Should it be eliminated by cancelling the attraction between the particles or making the particles mutually repellent? In other words, is cement dispersion a desirable end to be sought?

We have been told repeatedly over a number of years that the normal flocculated state of cement paste is undesirable. This conclusion has been set forth positively as if it were a proven fact. I hold that it is nothing but an unsupportable hypothesis.

Most of the data that have been presented in support of the claims made for cement dispersion actually show the combined effects of: 1) an accelerator, 2) air-entrainment and 3) a quantity of dispersing agent too small to eliminate interparticle attraction in concrete mixes. I hold that such data are not applicable to the question before us and do not therefore provide an answer to the question.

We will now consider some of the criticisms of the paper. As a device toward keeping the subject in sharp focus, we shall imagine that I am confronted with an interrogator who has read the printed discussions and noted the questions raised by the discussers. The hypothetical questions and their answers follow.

Q. In his printed discussion Dr. Scripture says, "This attempt to discredit cement dispersion as a factor in concrete mixes is based on an arbitrary *definition* and a process of reasoning from well known phenomena obtainable in dilute suspensions and assumptions unsupported by experimental evidence." Elsewhere in his discussion he says or implies that an accepted definition of "dispersion" already existed and that the definition used was chosen solely as an expedient toward supporting a prejudice. Must we accept this fundamental criticism of your paper?

A. Since this criticism, if valid, would discredit the whole paper, it seems justifiable to discuss it at some length. Dr. Scripture's basic criticisms can be accepted only by ignoring pertinent facts. We shall consider first some of the facts pertaining to the question of definitions.

In Research Paper No. 35 of the Master Builders Company, Dr. Scripture said, "The action of the dispersing agent is caused by its orientation with respect to the solid particles whereby these particles are endowed with electrostatic charges of like sign so that they are mutually repellent and do not stick together." He thus regarded the state of dispersion as one in which interparticle attraction is negative, that is, one in which the particles are mutually repellent. The definition that I used says, in effect, that a state of dispersion is one in which interparticle attraction is absent or too weak to be manifested in ways

described in the paper. This definition would include a state wherein the attraction is zero or negative, that is, one in which the particles are neutral or mutually repellent. Since this definition does not differ fundamentally from one used by Dr. Scripture himself and covers the state illustrated in his published photomicrographs, the reason for his rejection of it may not be readily apparent.

A reason for his rejection of the definition is suggested, however, by various other passages from his Research Papers and from published advertisements of the product called "Pozzolith," sold as a cement dispersing agent. From these publications it is apparent that Dr. Scripture wishes to define dispersion as something having an effect identical with the effect of Pozzolith. Such a definition has some obvious and some less apparent shortcomings. The less apparent ones will now be pointed out.

During 1941 and 1942, the years that Research Papers 35 to 39 of the Master Builders Company were issued, we analyzed several samples of Pozzolith. These samples contained the equivalent of 65 to 74 percent flake calcium chloride by weight, about 15 percent mineral matter, and the rest organic matter. All these samples, as well as every other sample tested since 1935, caused air entrainment. Thus, whatever effects this mixture might produce, they cannot properly be ascribed to cement dispersion alone. Moreover, as was pointed out in the discussion by Ernsberger and France, data on the results obtained from its use cannot properly be offered to illustrate the effects of cement dispersion.

Since Dr. Scripture does submit such data as proof of the effectiveness of dispersion, we may ask how he was able to discover that the acceleration of hydration, for example, is due to cement dispersion and not to the well known accelerating effect of calcium chloride; or how he was able to ascribe an increase in frost resistance to cement dispersion and not to air entrainment; or how he was able to ascribe reduced bleeding to dispersion and not to calcium chloride or to air entrainment or both.

Contrary to Dr. Scripture's statement it *was* necessary to establish a restricted definition of dispersion, because there is no generally accepted single meaning of the term. The situation was well summarized by Fischer and Jerome, *Ind. Eng. Chem.* V. 35, p. 336 (1943): "Dispersion, a generic term, refers to a process, procedure, or state relating to heterogeneous systems of solids and immiscible liquids." Dr. Scripture was apparently not aware of this when he attempted to refute my statement that a state of flocculation is essential to plasticity. He quoted Lewis, Squires, and Broughton, who said, "Dispersion . . . is essential to plasticity." A reading of the section of the book from which this quotation was taken will show that those authors were not refer-

ring to dispersion in the sense of deflocculation. Instead, they referred to the *process of mixing* fine particles, in this case clay, with water. In the fifth sentence of the same paragraph in which the above quotation appears they say, "There is every reason to believe that, when adequately dispersed, the surface of each ultimate particle is highly hydrated even where agglomeration (flocculation) is considerable." Actually, Lewis, Squires, and Broughton do not deny the fact recognized by many students of this subject, that the granular suspensions exhibit plasticity only when interparticle attraction is sufficiently strong. (See later.)

It seems a pity to take so much space to point out the futility of a discussion of dispersion wherein no two discussers attach precisely the same meaning to the term. Professor Staley, as well as Dr. Scripture, insists on using the term "dispersion" to describe a condition that is specifically treated in the paper but not called "dispersion". Had the state they wished to discuss been ignored in the paper, there would have been some excuse for ignoring my terminology. As it is, to reject my definitions serves no purpose other than that of hopelessly confusing the discussion. This is further illustrated in Mr. Weymouth's discussion. He calls particles that fall independently "dispersed" and others *in the same suspension* that do not fall independently he considers "flocculated", overlooking the explanation of that phenomenon given at the top of p. 123 of the paper.

Disagreement arises when each observer attempts to define dispersion in terms of whatever manifestation of interparticle attraction (or lack of it) comes to his attention, just as disagreement arose among the three blind men describing an elephant. The various manifestations of interparticle force were considered when writing the original paper, and attempts were made to write definitions in terms of one or more of them. Each such attempt led to difficulties of the same kind that now appear when the paper and the various printed discussions are considered together. The definition offered at the bottom of p. 123 makes it possible to discuss all these phenomena without confusion of terms.

We may consider next Dr. Scripture's basic criticism that my reasoning, and hence the conclusions reached in the paper, are based upon observations of phenomena obtainable only in dilute suspensions rather than in pastes, and upon assumptions unsupported by experimental evidence. From his remark that conclusions were drawn without supporting data or from data on dilute suspensions only, it seems that Dr. Scripture did not find time to examine the literature cited in the paper. The fact is that our conclusions were based on nearly 10 years of study of cement pastes, thick and thin, fresh and hard. Most of the results of this work have been published. The effects of flocculation

were deduced mainly from studies of pastes of the compositions that occur in concrete.

Thus we must categorically deny the validity of Dr. Scripture's basic criticisms. If these are rejected, together with Pozzolith data offered as data on dispersion, the superstructure of the criticism collapses also.

Q. Why do you say that cement is highly hydrophilic and needs no wetting agent when according to Professor Staley and M. Spindel the opposite is or may be true?

A. I am rather certain that both critics are in error; at least, no one has produced any evidence that after concrete is mixed in a concrete mixer, or even by hand, the individual grains are not completely surrounded with water. If the supposed unwetted condition were at all prevalent, it should be detectable microscopically on sections of hardened paste or concrete, for the unwetted portions would not hydrate appreciably. Within the first minute or so after introducing cement into water, chemical reaction occurs at a higher rate than in any subsequent period (William Lerch, "The Influence of Gypsum on the Hydration and Properties of Portland Cement Pastes," *Proc. ASTM*, V. 46 (1946), Bull. 12, P.C.A. Research Laboratory). This reaction produces an exceedingly thin film of gel (not a thick one as supposed by Mr. Schmid), which is pretty likely to be wet since it came into being within the body of the liquid. As a matter of fact, the water is strongly adsorbed by the solid paste. The affinity is so strong that the layer of adsorbed water has a density about 10 percent higher than that of free water; it is so strong that either fresh cement or dried hydrated cement makes an excellent agent for removing water vapor from air.

In his discussion of wetting Professor Staley fails to distinguish between spreading-wetting described in my paper and wetting by immersion. Dry cement is partially covered with a small amount of adsorbed air, an incomplete layer not over one molecule deep. Water, because of its greater attraction to the solid, will spread over the particles and displace the air (though not instantaneously). We proved this by tests on dry, hydrated cement (which holds much more air than fresh cement) in which we compared the amounts of water taken up by thoroughly evacuated samples with that taken up by samples known to contain adsorbed air.

The phenomenon of wetting does involve liquid adsorption as stated by Professor Staley, but Professor Staley is incorrect in stating that adsorption will take place "if . . . the force of adhesion between the solid and the liquid is higher than the force of cohesion of the liquid." He seems to regard a wetting agent as one which when dissolved in the liquid changes the intermolecular cohesion of the liquid. This

is hardly in accordance with present-day concepts of the phenomenon. The intermolecular cohesion in the liquid seems to have little to do with the adsorbability of the liquid. The force of cohesion among water molecules, for example, is much higher than between benzene molecules; yet water is strongly adsorbed by hydrated cement (and mineral oxides in general) but benzene hardly at all. Professor Staley is apparently thinking of surface tension when he speaks of cohesion, but even so there is no direct connection between adsorbability and surface tension.

Q. Since it has been proved by France and Ernsberger that calcium lignosulfonate, a dispersing agent, endows cement particles with a negative electrostatic charge, why do you contend that agents containing this substance do not disperse cement in concrete?

A. Direct observation shows that when the agent is used in the recommended quantities, the cement particles remain mutually attractive as shown by the fact that they form flocs in dilute suspensions; they are not rendered repellent. According to the definition adopted for the discussion, this denotes a state of flocculation, not dispersion. When a dilute suspension is made with a paste taken from such concrete, by filtering out some of the "cement solution" and mixing a little of the cement with the solution, flocs are clearly visible. In thick pastes the state of flocculation is manifested by the nature of the bleeding characteristics and the force-flow relationships.

Besides the direct indications just mentioned, we should remember also that the existence of an electrostatic charge on the particles does not in itself indicate a state of dispersion. It is a fact (apparently not known to Dr. Ernsberger and Professor France) that the cement particles in normal flocculated pastes carry a positive electrostatic charge. This charge was found and measured by Harold H. Steinour, who used the technique of electroosmosis instead of electrophoresis. Particles carrying a positive charge are undoubtedly mutually repellent to some degree. Yet we all agree that the particles in normal pastes show mutual attraction. As pointed out in the paper, the forces of attraction and repulsion may coexist. The flocculated state exists when the forces of attraction are greater than the forces of repulsion. It is the small positive electrostatic charge, together with a layer of adsorbed water—the lyosphere, that keeps the particles slightly separated even while they are held together by van der Waal's forces. (See next to last paragraph on p. 120.)

It is evident that to produce a state of dispersion by a negative colloidal ion, enough of the ion must be adsorbed to neutralize the original positive charge and to build up a negative charge sufficient to more than offset the attraction between the particles. The data of Ernsberger and

France indicate that the concentration required for complete cancellation of interparticle attraction or virtually so is that which will permit the adsorption of 0.5 g. of the agent per 100 g. of cement. We estimate that the samples of commercial materials with which we have worked carry about ⅕ to ⅓ the amount of dispersing agent required for dispersion when used in recommended amount. However, as brought out in the paper, the amounts present are usually sufficient to reduce the intensities of the force of flocculation. This reduction is among the effects that Dr. Scripture apparently prefers to call "dispersion" and what Professor Staley refers to as "partial dispersion", despite the fact that the particles remain flocculated.

Q. Why do you introduce data on sedimentation of emery particles or silica when, as mentioned by Professor Staley, these are obviously much different from portland cement and are much simpler?

A. These data were introduced because several years of study by both Steinour and me produced proof that the sedimentation of portland cement during the period when the paste remains plastic follows the same laws as those materials and many others. Even a suspension of tapioca in lubricating oil followed the same law as portland cement and other mineral powders in water. The proof is given in the published papers referred to.

Q. Dr. Scripture points out that you introduce data on dilute suspensions when at the same time you contend that pastes behave differently from such suspensions. What justification is there for this?

A. Dilute suspensions provide a ready means of observing the relative amount of interparticle attraction in a given medium, provided the effects of attraction are sufficiently pronounced. In the paper, data from dilute suspensions are used for this purpose only. They are not used to show the effects of interparticle attraction on cement pastes or other thick suspensions.

Q. If as you say either dispersion or a reduction in interparticle attraction increases the amount of bleeding, how do you account for the data presented by Dr. Scripture showing the opposite result?

A. Dr. Scripture presented data showing the effect of Pozzolith. If the Pozzolith that he used was like that we have analyzed, it was composed of about ⅔ part calcium chloride, ⅕ part dispersing and air-entraining agent, and the rest mineral filler. The effect on bleeding that he ascribed to dispersion has been produced in this laboratory by calcium chloride alone.

It may be noted that Mr. Schmid also believes that he has data showing that dispersion reduces bleeding. However, he too cannot be certain that the effect observed was due to dispersion per se. From the extensive studies of sedimentation made in this laboratory, particularly

those published by Steinour, we have ample support for our conclusions concerning the effects of interparticle attraction on bleeding.

Q. You say that cohesiveness and plasticity of cement paste is due to interparticle attraction. Dr. Scripture says that it is due to the attraction between the particles and the surrounding liquid.* Which explanation is correct?

A. This question can be settled first by directly observing the effect of reducing interparticle attraction in the absence of air entrainment. If you will make such an observation, you will find, as did Mr. Schmid, that cohesiveness is lacking. Linking the particles together by attractive forces cannot help but create cohesiveness in the mass as a whole and weakening or cancelling those forces must have the opposite effect.

Dr. Scripture refers to experience showing that a dispersing agent increases cohesiveness. But again he refers to Pozzolith, which entrains air. Entrained air increases cohesiveness.

Wiler and I studied the flow properties of paste and mortar (Ref. 7 of the paper). We found that even though the grains of cement are separated by films of water, a paste or a mortar exhibits structural rigidity, i.e., a yield value. In fact, a paste has some of the properties of a gel. It is slightly elastic and able to hold its shape against small forces. This gel-like behavior is the essence of plasticity and it is the result of interparticle attraction. The properties we call cohesiveness, fatness, or unctuousness are due largely to these forces. We have made concrete mixes with non-flocculated pastes and found that satisfactory workability could be obtained only by adding a flocculating agent.

Dr. Scripture says that the statement "Flocculation is essential to plasticity of granular suspensions" is a sweeping generalization that is manifestly incorrect. Students of this subject do not agree with him. Speaking of paints, Henry Green said, "The flocculated pigment is the 'structure' which holds the mass together giving it a plastic nature; when this structure is destroyed, plasticity tends to vanish . . . . . It is not necessary to remain content with this statement as pure theory, for it is easily verified as fact."† In another, more general paper‡ he illustrates suspensions of 5 different characteristics and says that plastic suspensions are always found to be flocculated. Paul S. Roller said, "It is the excess free space associated with the flocculated state that makes plastic flow possible . . . "§

A reading of these and various other papers, and especially Houwink's book *Elasticity, Plasticity and Structure of Matter* (Cambridge University

* See also, "Application of the Principle of Cement Dispersion to Portland Cement," by Edw. W. Scripture, Jr., Research Paper No. 35, p. 11 and footnote, Master Builders Research Laboratories, Cleveland, Ohio.
† *Ind. Eng. Chem.* V. 15, p. 122 (1923).
‡ *J. Appl. Phys.*, V. 13, p. 611 (1942).
§ *J. Phys. Chem.*, V. 43, p. 457 (1939).

Press, 1937) will show that dispersed (deflocculated) suspensions are never plastic. When dilute, they are fluid; when concentrated, they are dilatant.

Q. You admit that dispersion or a reduction in interparticle attraction reduces the water requirement for concrete of a given slump. Yet you say that workability may not be improved by the use of a dispersing agent. Are not these statements contradictory?

A. The statements are not necessarily contradictory. If two mixes made with the same aggregate and cement have the same slump but different paste contents, the one with the lower paste content will be the less workable, at least under circumstances where high plasticity is required. The practical aspects of the principles involved are well illustrated by the photographs in Fig. F and G. In Fig. F the sample at the left represents a plain concrete mix containing about 5 sacks of cement per cubic yard of concrete, slump 1.2 in. In the center is the same mix containing the same amount of water but with an agent that reduced the attraction between the cement particles. The slump was 7.5 in. Although the mix with the added agent could undoubtedly be placed with less effort than the one without the agent under some cir-

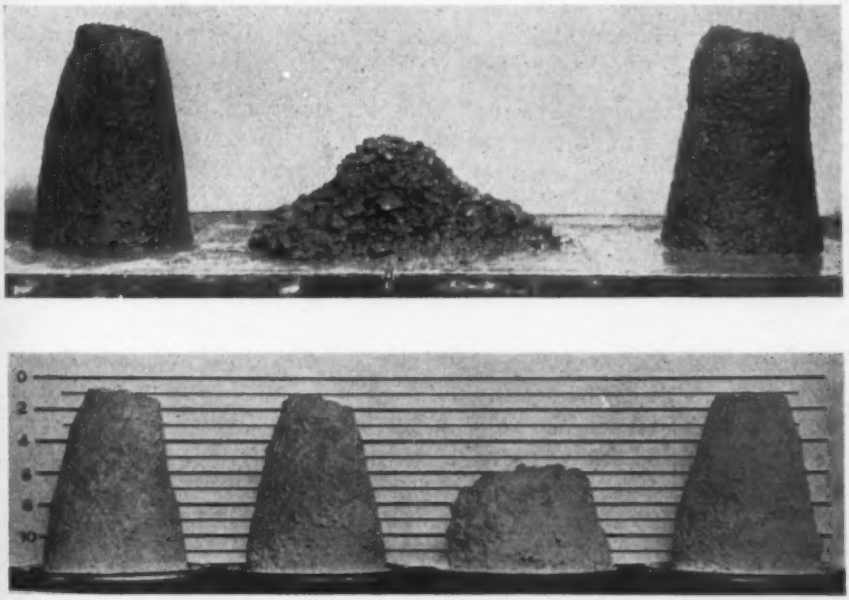

Fig. F (top)—The effect of an agent that reduces interparticle attraction without entraining air

Fig. G (bottom)—Effect of air entraining agent that also reduces interparticle attraction

cumstances, it is clear than an increase in slump does not signify an improvement in plasticity and cohesiveness.

At the right we see the effect of adding the agent and reducing the water content by 19 percent. It is very apparent that this mix is not as plastic as the mix without the agent (at the extreme left).

Fig. F was referred to by Mr. Schmid.* It should be noted that he makes no claim that dispersion improves workability. He merely says that dispersion (or at least Agent A) shows no ill effects if used in mixes sufficiently rich.

In connection with the general question of workability attention is called to Fig. G. The two samples on the left represent normal concrete mixes with about 5⅓ sacks of cement per cu. yd. made with high early strength and Type I portland cements, respectively. The third sample represents a sample made with a cement containing an agent that reduced interparticle attraction and at the same time entrained a small amount of air. This change obviously represents a marked increase in plasticity. The contrast between this and the effects shown in Fig. F is obvious.

The fourth sample also represents concrete with the added agent but with the paste content reduced by increasing the aggregate while keeping the cement and water in the same ratio. The mix still appears to be plastic; whether it is as plastic as the richer mixes without the agent is debatable.

These observations show our basis for doubting that a weakening of interparticle attraction in the absence of air entrainment is desirable. We believe that under most circumstances where workability is lacking the kind of improvement desired is that which is obtained when the paste is stiffened and the paste content of the mix is increased. Such is the effect of adding mineral powder such as hydrated lime or diatomaceous earth. Such additions increase the capacity of the mix for plastic deformation and increase cohesiveness. An agent that softens the paste brings about the opposite change; that is, to maintain a given consistency of the concrete the paste content must be reduced because the paste is softer. When the paste quantity is reduced, the average spacing of the aggregate particles is reduced and the plasticity of the mix is correspondingly diminished. This constitutes a reduction in workability which is tolerable only in mixes originally having more than sufficient plasticity.

Q. Mr. Forbrich gives data indicating that a dispersing agent is a desirable aid in mixing. Should this not be considered at least a point in favor of the use of such agents?

*See Fig. C, p. 140-30.

A. When working with neat cement we have observed effects like those described by Mr. Forbrich. However, with concretes and mortars comparable effects were not detectable. We attribute this to the vigorous kneading effect of the aggregate. This effect is so strong as to obviate the necessity for any mixing aid.

Q. The proponents of the use of dispersing agents, Dr. Scripture among them, state that in its normal state of flocculation the hydration of portland cement is inhibited by the inability of water to enter the clusters of cement particles. Does flocculation inhibit hydration?

A. Over 30 years ago Nathan C. Johnson* exhibited photomicrographs to an A.S.T.M. audience showing what he thought were large clusters of unhydrated cement grains in well cured briquet and concrete specimens. He sa[illegible] cement grains become clustered as they become wetted and [illegible] clusters thus become permanently sealed against subsequent hydration. He believed that over 80 percent of the cement remained unhydrated because of this effect. However, in the ensuing discussion P. H. Bates and H. S. Spackman showed that what Mr. Johnson had thought were clusters of cement grains was in each case a single unhydrated cement grain; that is, what he thought were grain clusters were exposed crystalline constituents of the single grain.

So far as I know Mr. Johnson's has been the only published attempt to prove by experimental evidence that flocculation inhibits hydration. Since that time the statement has been made repeatedly, but no proof has been offered.

There is, however, direct proof that hydration is not so inhibited or at least not prevented. Attention is called to photomicrographs published by Dr. Brownmiller† in 1943. These photomicrographs showed high early strength cement to be almost completely hydrated in seven days and a Type I cement to be about 85 percent hydrated in 28 days. It should be noted that the unhydrated material always appears as isolated, large grains. All the smaller surrounding grains have disappeared. Other studies made in this laboratory, and now being published currently in this journal, support this indication that under conditions of continuous wet-curing all the particles in normal cement pastes except those that originally had diameters in excess of about 50 microns become completely hydrated.

The current notion that a large part of the cement remains unhydrated is a carry-over from observations made in earlier days when cements were coarsely ground. It is also the result of observations made on pastes of normal consistency. Such pastes do not become hydrated to the same extent as those having water ratios in the practical range.

*Proc. A.S.T.M., V. 15, Part II, p. 172 (1915).
†Proc. ACI. V. 39, p. 193 (1943).

Various points of difference other than those mentioned above were raised by the discussers. The underlying arguments are usually based on the results obtained from commercial admixtures that, as used, modify the state of flocculation more or less but do not cause dispersion as defined in this paper or as originally defined and illustrated by Dr. Scripture. Since they do not cause dispersion so defined, their effects cannot properly be ascribed to dispersion.

In other instances objections are based on differences in fundamental concepts that cannot be removed by argument alone.

* * * * *

In retrospect we can see that the question, "Should portland cement be dispersed?" is a different one from, "Should products sold as dispersing agents be used in the manufacture of concrete?" The answer to the first depends on the validity of the assumption that interparticle attraction and its effects are undesirable. The answer to the second depends on the actual performances of the products and not on the validity of the theories that have been set forth in connection with those products.

It is believed that the first step toward determining the true value of these products and their ultimate place in the industry is to abandon the theory that the flocculated state is undesirable. The notion that flocculation inhibits hydration or that dispersion accelerates it should be cast aside. The assumption that cohesiveness and bleeding characteristics can be improved by cancelling or reducing interparticle attraction should also be discarded. To hold these notions in the face of contrary evidence now available will do nothing toward advancing the art of concrete construction.

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An Investigation of the Strength of Welded Stirrups in Reinforced Concrete Beams*

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Member American Concrete Institute

SYNOPSIS

The results of the tests of 44 beams of reinforced concrete with stirrups welded to the longitudinal reinforcement are presented. The beams were designed in such a way as to produce failure by diagonal tension. Variables including the size and inclination of the stirrups, type of concrete and ratio of longitudinal reinforcement are studied. A comparison of the strength of welded stirrups with that of loose stirrups, as reported from former tests on web reinforcement, is attempted.

1—INTRODUCTION

The use of welded stirrups as web reinforcement for concrete beams, instead of the conventional loose U-stirrups, should provide improved construction, since it gives perfect continuity to the reinforcing steel embedded in concrete and facilitates accurate steel-setting. Its practical use in units formed by longitudinal bars and stirrups welded together has been suggested as economically possible.‡

Two main questions arise in connection with the problem: (a) Will beams containing stirrups rigidly connected to the main reinforcement develop higher shearing strengths than those using common loose stirrups? (b) Can the higher theoretical efficiency of certain inclined stirrups be utilized through the use of welded units?

A favorable answer to these questions is important, since the economical use of welded stirrups will depend to a great extent upon the advantages to be secured by the use of such web reinforcement.

*Submitted to the Institute March 31, 1945.
†Graduate Fellow in Civil Engineering, University of Illinois.
‡D. M. McCain, "Welded Shear Reinforcing for Concrete Beams," *Civil Engineering*, July, 1939.

Probable advantages in strength due to welded stirrups lie in improved anchorage of the stirrups, prevention of slipping of inclined stirrups along the main bars and improved anchorage of the main bars.

The advantage to be secured by the use of certain inclined stirrups is indicated theoretically, as shown by the following summary of the usual analysis of web stresses. In such analysis, it is common to assume that the action of a reinforced concrete beam may be likened to that of a truss in which the top chord is formed by the compression zone of the concrete, the bottom chord by the longitudinal reinforcement, the tension web members by the stirrups, and the compression web members by portions of the concrete web of the beam. The assumption is also made that the compression web members are inclined at 45 deg. to the axis of the beam. The analysis shows that the strength of stirrups having a given volume of metal per unit volume of concrete depends on the inclination of the stirrups with respect to the axis of the beam.* Also according to the analysis, vertical stirrups and stirrups at 45 deg. are equally strong. The maximum strength is given by stirrups inclined at 67.5 deg. If the strength of the beam with vertical stirrups or stirrups inclined at 45 deg. is taken as unity, the analysis shows for stirrups inclined at 67.5 deg. a beam strength about 20 per cent greater. The inclination to be given to the welded stirrups should be the one that gives the maximum strength.

This paper contains the results of the tests of 44 beams of reinforced concrete, loaded at the third points with a span of 8 ft., and designed to produce diagonal tension failure. The tests were intended to give results showing the strength of welded stirrups and to point out which is the best inclination, with respect to the axis of the beam, to be given to those stirrups. The following variables were introduced in the investigation: (a) type of concrete, (b) ratio of web reinforcement, (c) inclination of the stirrups, and (d) ratio of main reinforcement, which was varied in a supplementary series of tests on four beams. These four beams are treated separately throughout this paper and are referred to as Series 1a. A comparison is also made with former tests on loose stirrups.

Notation

The following notation is used, particularly in the headings of Tables 1 to 3:

f'_c = average compressive strength of 6 by 12-in. concrete control cylinders (3 for each beam)

f_y = yield point stress of web reinforcement

$p = \frac{A_s}{bd}$ = steel ratio of longitudinal tension reinforcement

*F. E. Richart, "An Investigation of Web Stresses in Reinforced Concrete Beams," University of Illinois, Bulletin No. 166.

$r = \dfrac{A_v}{ab}$ = ratio of web reinforcement, where A_v is the area of the two single stirrups at a cross-section, a is the stirrup spacing, measured normal to the direction of the stirrup, and b is the width of the beam

k = ratio of the distance from the neutral axis to the top of the beam, to the effective depth d

K = $(\sin \alpha + \cos \alpha) \sin \alpha$ = 1.0 when α = 45 deg. or 90 deg., and 1.20 when α = 67½ deg.

α = the inclination of the stirrups to the axis of the beam

P_u = ultimate load on beam

V_c = vertical shear (one-half the load on the beam) at which diagonal cracks were first noted

V_u = vertical shear (one-half the ultimate load) at ultimate load

V_y = vertical shear (one-half of the load on the beam) at which yield point stress was reached in web reinforcement

$v_c = \dfrac{V_c}{bjd}$ = shearing stress at which diagonal cracks were first noted.

$v_u = \dfrac{V_u}{bjd}$ = shearing stress at ultimate load

$v_y = \dfrac{V_y}{bjd}$ = shearing stress corresponding to vertical shear, V_y

v_u *calc* = shearing stress v_u, calculated from Eq. 8.

v_y *calc* = shearing stress v_y, calculated from Eq. 6.

2—DESCRIPTION OF TESTS

Relatively few of the specimens heretofore tested to determine the strength of web reinforcement in beams of normal dimensions have had a definite diagonal tension failure. Initial failure by bond has played an important part in the behavior of those beams, resulting in a failure due to the simultaneous effect of bond and diagonal tension. Many of them have failed in tension, pointing out the necessity of using a fairly large amount of tension steel, when the web reinforcement is of some importance, in order to avoid this kind of failure.

The specimens used in this investigation were designed to produce a true diagonal tension or shearing failure and to avoid as far as possible all other influences. Care was taken to eliminate all variables except those being studied. The beams were all 10 ft. long, 5.5 in. wide, and 21 in. deep. Forty of the beams were reinforced longitudinally with four 1-in. square bars placed in two layers, with an effective depth of 18.25 in. The other four beams were reinforced with two 1-in. square bars, with an effective depth of 19.5 in.

Three series of beams were tested: two groups, each of twenty beams, were identical except for the type of concrete. In Series 1, Mix No. 1 (1:3.34:5.0, by weight, with a water-cement ratio of 0.75 by weight) was

TABLE 1—OUTLINE OF BEAM TESTS

Two companion beams of each type shown. Total 44 beams. Stirrup spacing 6½ in. normal to direction of stirrup. All bars of intermediate grade, deformed, except ¼-in. bars which were plain. Portland cement concrete; aggregates, sand, and gravel from Wabash river. Tension reinforcement, 1-in. square bars, having average yield point stress of 48,000 psi.

Series No.	Designed Concrete Strength psi.	p, per cent	r, per cent									
			None	45 deg. Incl.			67.5 deg. Incl.			90 deg. Incl.		
				¼–Ø	⅜–Ø	½–Ø	¼–Ø	⅜–Ø	½–Ø	¼–Ø	⅜–Ø	½–Ø
1	3500	4.00	0	0.28	0.615	1.12	0.28	0.615	1.12	0.28	0.615	1.12
2	4500	4.00	0	0.28	0.615	1.12	0.28	0.615	1.12	0.28	0.615	1.12
1a	3500	1.88	.	....	.....	.....	....	.....	....	0.28	0.615	

used. In Series 2, Mix No. 2 (1:2.79:4.18, by weight, with a water-cement ratio of 0.64) was employed. In the third group, Series 1a, consisting of four beams, Mix No. 1 was used. Table 1 lists the design features of the test beams, and Fig. 1 shows the general dimensions, details of reinforcement, and location of strain gage lines for these beams.

The longitudinal and web bars were first welded together in units, as shown in Fig. 2a and b. Two units, forming the complete reinforcement for a beam, were then joined at the proper distance by welding on small transverse spacing bars, as shown in Fig. 2c.

The beams and control cylinders were cured for 21 days in the moist room, then were stored in the air of the laboratory until they were 28 days old, when they were tested.

Strain measurements were taken on 21 gage lines (19 in Series 1a), located as shown in Fig. 1. Sixteen of these gage lines were on the stirrups, eight on each side of the beam, located symmetrically. The rest of the gage lines were used to measure deformations in the main reinforcement and in the concrete at midspan. A Berry strain gage, of 5-in. gage length, was used for this purpose. The ultimate load was reached in six to ten increments, a complete set of readings being taken at each increment. The deflections of the beams, and the load at which the diagonal cracks started, were also recorded during the progress of the tests.

In designating the beams, the following notation was used: The first number (1, 2, or 1a) indicates the series. The following letter specifies the type of web reinforcement: N, no stirrups; V, vertical; I, 67.5 deg.

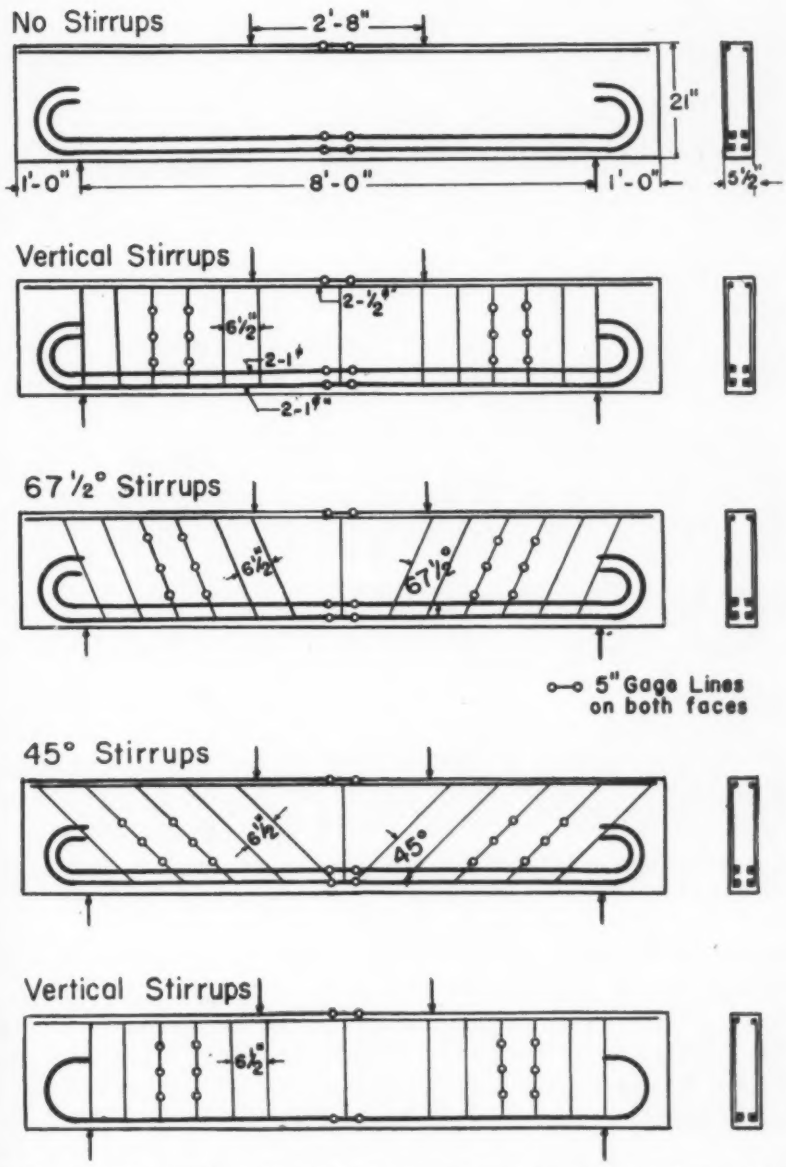

Fig. 1—General dimensions, details of reinforcement and location of strain gage lines of beams.

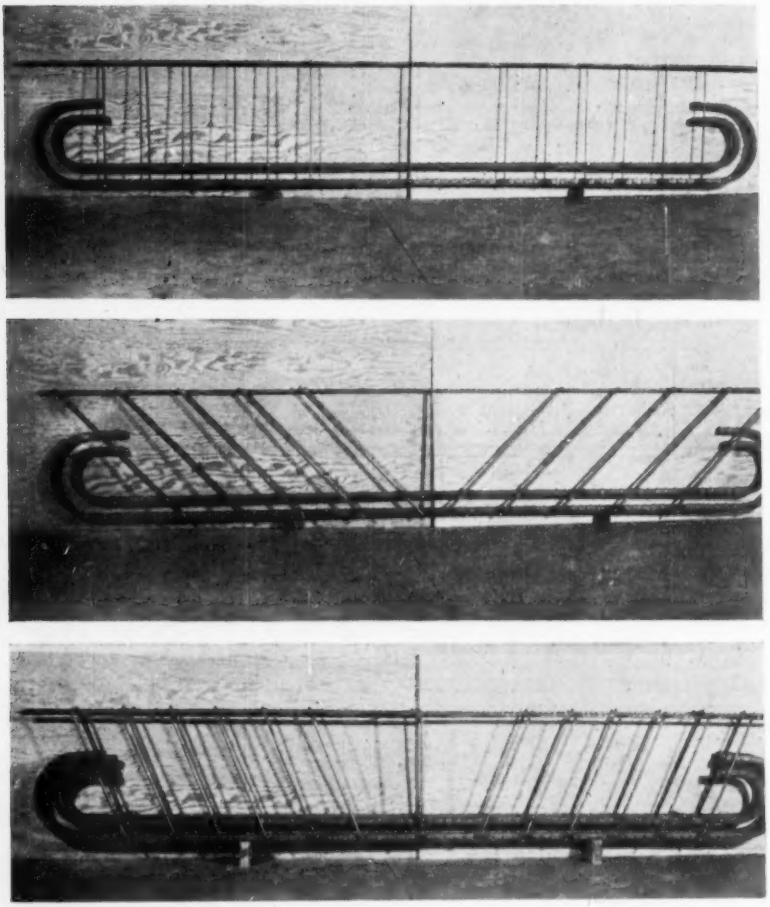

Fig. 2—View of the reinforcement of beams. (a top) Unit with vertical stirrups, (b center) Unit with 45 deg. stirrups, (c bottom) Complete reinforcement of a beam with 67.5 deg. stirrups.

inclination; D, 45 deg. inclination. The fraction represents the diameter of the stirrups used. For example.

1-N indicates: Series 1, beam without web reinforcement.

2-D ½ indicates: Series 2, ½-in. stirrups at 45 deg. inclination.

1-I 3/8 indicates: Series 1, 3/8-in. stirrups at 67.5 deg. inclination.

1a-V ¼ indicates: Series 1a, ¼-in. vertical stirrups.

The phenomena of the tests showed the following general picture: Up to a load of about 50,000 lb., corresponding to a computed shearing stress of 300 psi, no diagonal cracks were developed. The stresses in the stirrups were very small, the diagonal tension being taken up by the concrete, which was still uncracked.

For the beams without web reinforcement, after the diagonal cracks started, a main crack developed on each side of the beam. These cracks, which were inclined at about 45 deg. to the horizontal, started in the lower half of the beam and progressed upward as the rate of deflection of the beam increased. At ultimate load, the upper portion of the beam sheared off in the compression zone.

For the beams with web reinforcement, the diagonal cracks started in the lower half of the beam as before. As the load increased, these cracks extended upward, while new cracks that started at the bottom of the beams extended vertically for a while, then turned at about 45 deg. and ran approximately parallel to the other diagonal cracks. The number of cracks developed and the distance between them was, on the average, dependent on the ratio of web reinforcement. At ultimate load, the beams with ¼-in. stirrups had a main diagonal crack on each side, at each end, with relatively few cracks parallel to these. In the beams with 3/8-in. stirrups, the diagonal cracks were more numerous and the distance between them was, on the average, smaller. All beams with ½-in. stirrups failed in compression when the stress in the web reinforcement had barely reached the yield point, so that the diagonal cracks were not fully developed; however, at ultimate load, the number of cracks was generally greater than for the other beams.

In general, almost all of the diagonal cracks were well started at the load at which the web reinforcement was stressed to the yield point. Increasing the load merely opened up these cracks, extending them upward until the ultimate load was reached by shearing off the upper portion of the beam.

All the beams failed by diagonal tension except those with ½-in. stirrups, in which failure was due to crushing of the concrete in compression, and those of Series 1a with 3/8-in. stirrups, in which failure was due to the simultaneous effect of tension in the main steel and diagonal tension. For all beams that failed by diagonal tension, failure occurred in about the same way. One of the two main cracks at each side of the specimen, which had extended over about 7/8 of the depth, started to open up rapidly at mid height as the stirrups yielded, developing at both ends until the remaining uncracked concrete at the top of the diagonal crack failed by shear, causing a sliding of the two parts of the beam along the crack.

3—SERIES 1 AND 2, ANALYSIS OF TESTS

Experimental data

Table 2 gives properties of the materials used in the beams, together with the principal results of the tests. Fig. 3 gives average curves show-

TABLE 2—RESULTS AND ANALYSIS OF TESTS

(Two companion beams of each type shown)

Beam No.	f'_c psi.	f_v* psi.	k	v_c psi.	P_u lb.	v_u psi.	V_y lb.	v_y psi.	$v_u - v_y$ psi.	$v_y - Krf_y$ psi.	v_y calc. psi.	Ratio v_y calc. / v_y	v_u calc. psi.	Ratio v_u calc. / v_u
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
							Series 1 and 2							
1-N	3585			261	79,000	474								
2-N	4490			321	84,500	507								
1-V ¼	3460	55,000		300	116,600	699	44,705	536	163	382	492	0.914	700	1.001
2-V ¼	4805	55,000	0.507	339	135,100	810	45,100	541	269	387	546	1.012	834	1.065
1-I ¼	3795	55,000	0.555	300	120,950	725	44,555	534	191	349	537	1.006	765	1.054
2-I ¼	4700	55,000	0.516	312	132,000	792	46,450	557	234	372	572	1.027	854	1.086
1-D ¼	3475	55,000	0.541	282	118,550	711	42,180	506	205	352	492	0.972	701	0.985
2-D ¼	3455	55,000	0.578	318	140,550	843	44,520	534	309	380	492	0.924	699	0.830
1-V ⅜	3115	47,900	0.565	345	146,850	881	54,450	653	228	358	624	0.955	810	0.919
2-V ⅜	4120	47,900	0.535	300	143,650	862	53,860	646	215	351	660	1.022	907	1.054
1-I ⅜	3575	47,900	0.559	378	159,000	954	60,855	730	223	376	697	0.949	911	0.955
2-I ⅜	4185	47,900	0.539	324	168,000	1008	59,915	719	289	364	721	0.992	972	0.969
1-D ⅜	2855	47,900	0.552	300	129,750	778	52,155	626	152	331	610	0.971	780	1.002
2-D ⅜	3685	47,900	0.578	300	143,650	862	53,185	638	223	338	642	1.007	863	1.002
1-V ½	3665	50,700	0.508	330	157,000	942	74,240	891	Compression Failure	323	915	1.027		
2-V ½	4815	50,700	0.535	267	186,400	1118	82,125	985		417	960	0.977		
1-I ½	3090	50,700	0.578	360	177,750	1066	83,080	997		315	1005	1.009		
2-I ½	4210	50,700	0.560	360	196,200	1177	89,280	1071		389	1050	0.980		
1-D ½	2705	50,700	0.569	315	155,000	930	74,850	900		332	876	0.974		
2-D ½	3495	50,700	0.549	360	175,825	1055	73,880	886		318	907	1.023		
							Series 1a							
1a-V ¼	3540	46,000	0.383	195	106,100	558	35,425	372	182	243	364	0.978	576	1.031
1a-V ⅜	3335	52,000	0.397	179	117,350	617	50,395	530		210	547	1.034		

*Determined by extensometer tests on samples of bars.

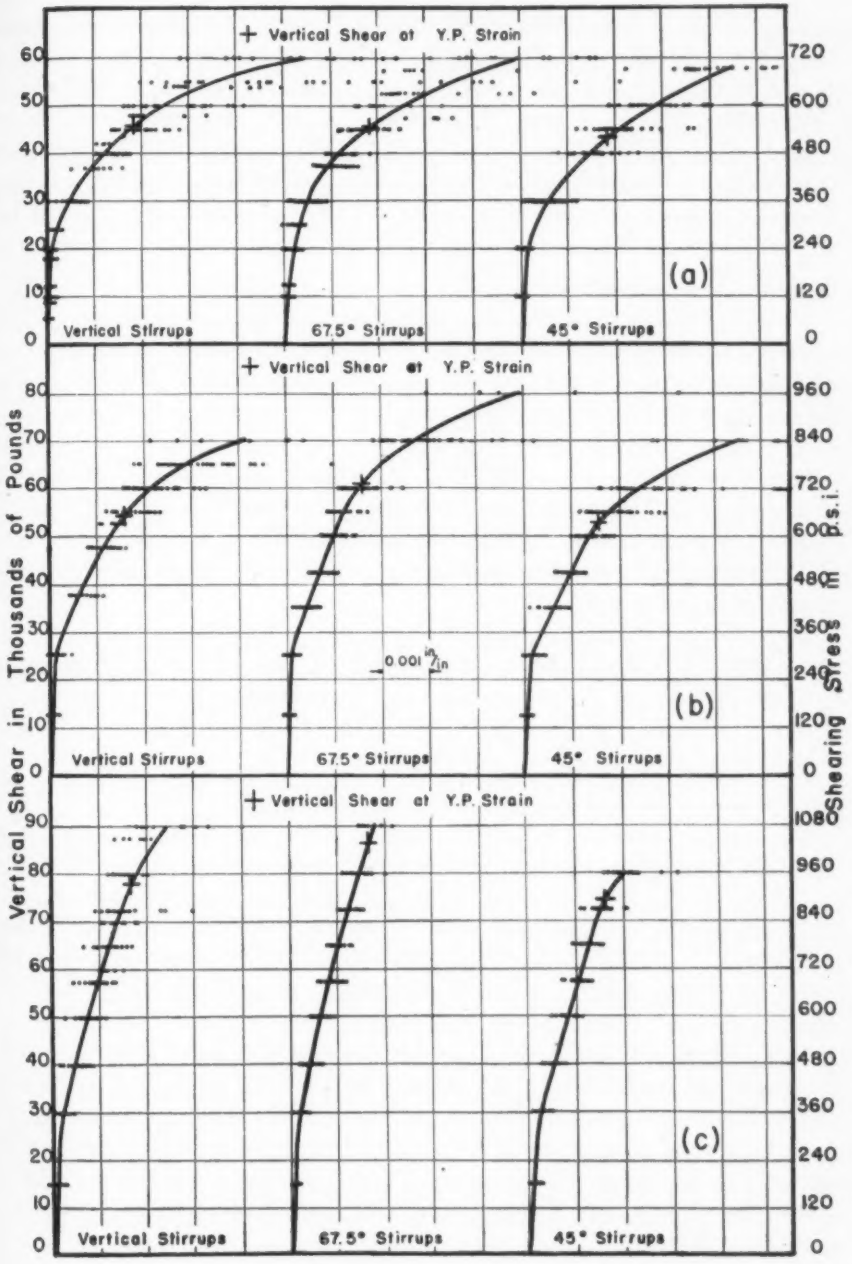

Fig. 3—Average 'shear-strain' curves for the web reinforcement. Series 1 and 2. a. ¼ in stirrups, (b) ⅜ in. stirrups, (c) ½ in. stirrups.

ing the relation between vertical shear and strain in the web reinforcement for each type of beams tested. In the right hand margin of each figure, the shearing stress corresponding to the vertical shear (computed as explained later) is shown. The plotted points in those figures represent actual values from the tests. In plotting these curves, and in analyzing the results, only about half of the measured strains (those which reached the yield point first) were considered. These were generally located on gage lines across or near the principal diagonal cracks.

Position of the neutral axis

From the flexural strains determined at midspan, the position of the neutral axis was computed for each loading. The values so determined, and corresponding to the last five increments of loading at which strain measurements were taken, were averaged and are given in Table 2 for each beam. As can be seen from this table, k ranges from 0.507 to 0.578 with a general average of 0.549. This figure corresponds practically to the value of k computed for the cracked section with $n = 10$. From $n = 10$, the value of $j = 0.833$ was determined for the cracked section and this constant value was used to compute the shearing stresses for the beams of these two series.

It is realized that this value of j may not be the actual value at the outer thirds of the beams, but it is in agreement with the usual practice in determining the shearing stresses. Furthermore, it is not believed that the lever arm at the outer thirds is very different from that at the center when diagonal cracks have developed.

Average shear at the yield point stress in the web reinforcement

The vertical shear at which the strain in the web reinforcement was equal to the known yield point strain for the steel was determined for each gage line. The smallest eight shears so selected were averaged. This average is arbitrarily defined as the "average shear at which the stress in the web reinforcement reached the yield point" and will be called V_y. This shear and the corresponding shearing stress have been selected as the basic values for the analysis of the tests. This selection and the consideration of that shearing stress as the basic value on which a permissible stress should be based is just one criterion for that purpose. Although all of the beams that failed by diagonal tension took more load than necessary to stress the stirrups to the yield point, this extra load was taken at the expense of an excessive increase in width of the diagonal cracks. It remains to be determined whether a shear V_y sustained indefinitely would not produce failure of the beam.

Method of analyzing results

In Table 2, the analysis of the results of the tests for these two series is presented. Columns 1 to 10 of this table give various quantities that

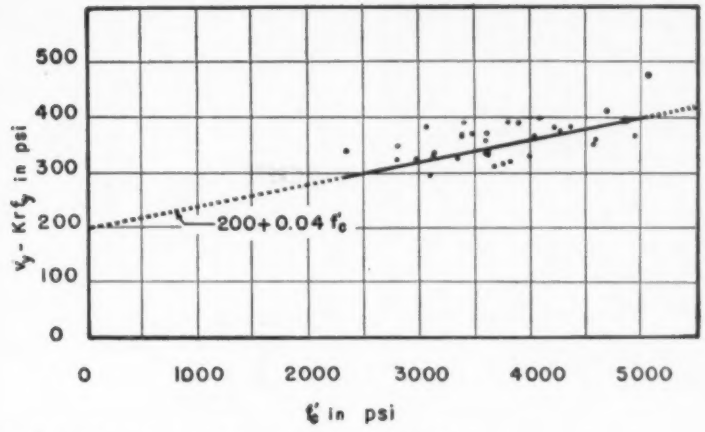

Fig. 4—Relation between v_y—Krf_y and concrete strength.

have been determined by the tests. They are self-explanatory or have been explained previously. In column 11 are given the values of:

$$v_y - Krf_y$$

Krf_y is the theoretical shearing stress at the yield point strength of the web reinforcement when the analogy with a truss is made in the analysis of the action of stirrups in reinforced concrete beams.*

The values of $v_y - Krf_y$ have been plotted in Fig. 4 against the concrete strength as determined from standard test cylinders. Through the points so determined, a straight line has been passed, for which the equation is

$$v_y - Krf_y = 200 + 0.04f'_c \text{ (in psi)}$$

The shearing stress at which the stirrups are stressed to the yield point, as defined previously, is thus represented fairly well, for these series of tests by the following equation:

$$v_y = Krf_y + 0.04f'_c + 200 \text{ (in psi)} \ldots\ldots\ldots\ldots\ldots (1)$$

The shearing stresses determined by this formula are shown in Column 12 of Table 2 and are denoted as "v_y *calc.*" Column 13 of the same table gives the ratio between the shearing stresses calculated by formula (1) and those derived from the experimental value V_y.

The calculated and experimental values of v_y agree more closely than could generally be expected for tests on reinforced concrete beams. When individual beams are considered, the ratio between calculated and experimental values ranges between 0.914 and 1.027. The general average for the beams with ¼-in. stirrups is 0.976, for those with ⅜-in. stirrups,

*See, for example, Bulletin 166, University of Illinois, loc. cit.

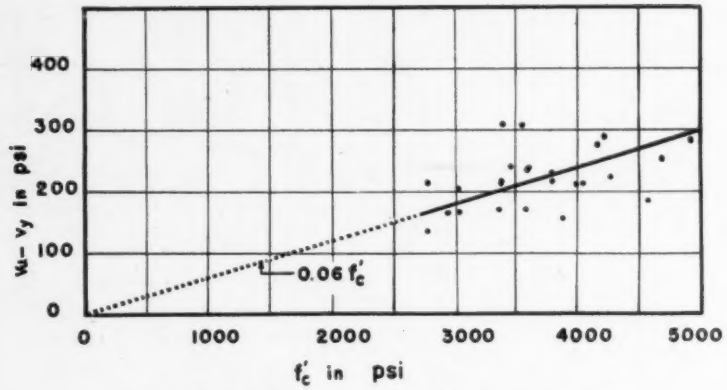

Fig. 5—Relation between v_u—v_y and concrete strength.

0.983, and for those with ½-in. stirrups, 0.998. The general average for all the specimens is 0.986.

Column 10 of Table 2 shows the difference between the experimental shearing stresses, $\frac{V}{bjd}$, at ultimate load and at V_y. This difference, $v_u - v_y$, has been plotted against concrete strength as abscissas in Fig. 5. A straight line has been passed through the points so determined. The equation of this straight line is:

$$v_u - v_y = 0.06f'_c$$

substituting v_y as given in equation (1):

$$v_u = Krf_y + 0.10f'_c + 200 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

With this formula, the shearing stresses at ultimate load have been computed for each beam, are given in Column 14 of Table 2 and are denoted as "v_u *calc.*" Column 15 gives the ratio between the shearing stresses computed by formula (2) and those determined experimentally. The agreement between calculated and experimental values is again very good, with ratios that range from 0.830 to 1.086 for individual beams. The general average for the beams with ¼-in. stirrups is 0.998, for those with ⅜-in. stirrups, 0.983, and the general average for all the beams that failed by diagonal tension, 0.99.

The measured deflections at the center of the beams were compared with the theoretical deflections computed by Maney's equation.* For a beam loaded at the third points, this equation is

$$D = \frac{23}{216}\frac{l^2}{d}(c + t)$$

*Maney, G. A., "Relation Between Deformation and Deflection in Reinforced Concrete Beams," Proc. A.S.T.M. Technical Papers, Vol. 14, p. 310, 1914.

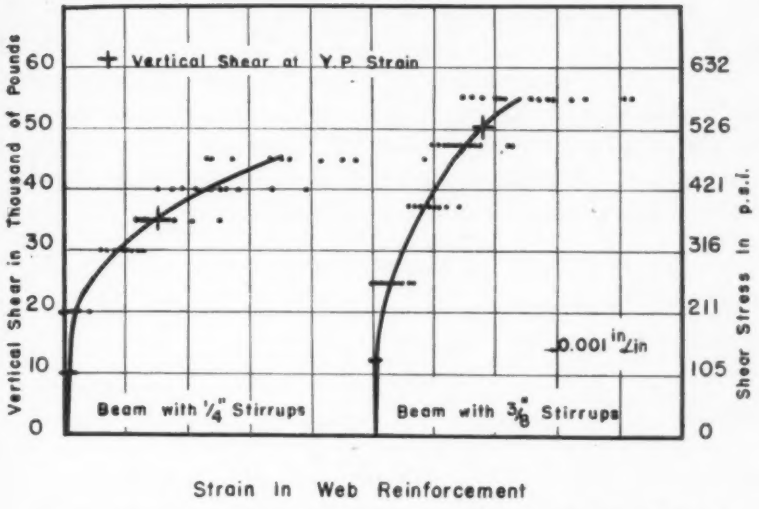

Fig. 6—Average "shear-strain" curves for the web reinforcement. Series 1a.

in which l is the span, d the effective depth, c the strain at the top fiber, and t the average strain in the tension steel.

The measured deflections at ultimate load were in certain cases as much as 40 per cent greater than the computed. At the load corresponding to V_y, the measured deflections were on the average 10 to 20 per cent greater than the calculated. Part of this disagreement between measured and calculated deflections is due to the effect of the shearing deflections, and the other part is believed to be due to the opening of the diagonal cracks, which Maney's equation does not take into account.

4—SERIES 1a, ANALYSIS OF TESTS

This series of tests was made with the purpose of studying the effect of varying the ratio of main reinforcement. The four beams of this series were entirely similar to those of Series 1 with vertical stirrups of ¼ and ⅜-in. diameter, the only difference being in the main steel. Table 2 gives the principal properties of the materials used in the beams, together with the results of the tests. In Fig. 6 are given average shear-strain curves for the two types of beams tested.

The analysis of tests was made in the same way as for Series 1 and 2. The lever arm of the resisting couple was determined for the cracked section with a value of $n = 8$ corresponding to the average experimental $k = 0.39$. With $n = 8$, $j = 0.886$. From an analysis similar to that explained for Series 1 and 2, the following general equations were fitted to the test results:

$$v_y = Krf_y + 0.04f'_c + 94 \text{ (in psi)} \quad (3)$$

and

$$v_u = Krf_y + 0.10f'_c + 94 \text{ (in psi)} \quad (4)$$

Table 2 shows the relation between the values calculated by the foregoing formulas and the experimental values. The agreement between calculated and experimental values is again very good with averages that for all practical purposes can be considered as 1.0.

5—DISCUSSION

The analysis of these tests indicates that the strength of welded stirrups can be given by a formula of this type:

$$v = Krf_y + qf'_c + \text{sp} \quad (5)$$

in which v is the shearing stress, Krf_y is the shearing stress that the beam would be able to resist if the truss analogy that is usually made in analyzing the action of the web reinforcement in reinforced concrete beams held absolutely true in an actual beam. The quantity qf'_c, in which q is a coefficient and f'_c the concrete strength, gives the effect of the concrete in resisting diagonal tension. The coefficient q for the beams tested was 0.04 when the shear V_y was considered, and 0.10 when the ultimate shear was analyzed. The quantity sp, in which s is a coefficient and p the ratio of tension steel, gives the effect of the main reinforcement on the resistance to diagonal tension. The factor sp was 200 in formulas (1) and (2), and 94 in formulas (3) and (4). With these values, the coefficient s is 5000.

Although the quantity sp has been taken as proportional to the ratio of main reinforcement, there is no definite proof from the tests for this supposition, which has been based mainly on the results of Series 1a. Should this assumption be true, it seems likely that the factor sp is not only a function of the ratio of reinforcement but also of the size of the bars used. No definite answers can be given to these questions, as only two ratios of main reinforcement and one size of bars were used in these tests.

The action of a beam with welded stirrups in resisting diagonal tension when it is subjected to a shear in the neighborhood of V_y may be likened to that of a truss in which the top chord is formed by the compression zone of the concrete, the bottom chord by the longitudinal reinforcement, the tension web members by the stirrups, and the compression web members by portions of the concrete web of the beam. The members of this truss are connected rigidly, but the rigidity of the tension web members is so small that they act practically as if they were pin-connected. The top chord, the bottom chord, and the compression web members have enough rigidity so that the truss can be thought of as a

truss whose behavior lies somewhere between that of a Vierendeel truss with special diagonal members and that of a pin-connected truss. The stresses that would exist in the tension web members if the truss were pin-connected are relieved by the rigidity of the other members. The factor qf'_c in formula (5) would account for the rigidity of the top chord and that of the compression web members, and the factor sp for the rigidity of the main reinforcement. The scattering of the points in Fig. 4 from the straight line would depend on variations of the rigidity of the top chord and compression web members due to differences in the extension and location of the diagonal cracks and in the distance between them.

After the shearing stress V_y is reached, the deformation of the tension web members increases so rapidly that the truss can take more load only at the expense of the compression chord whose flexural resistance is rapidly decreased by the extension of the main diagonal crack until the remaining part fails by shear along this crack. This would explain why the values of $v_u - v_y$ are a function of the concrete strength only.

The analogy usually made, in which the action of the beam is likened to that of a pin-connected truss, appears to be substantially correct in determining the value of the coefficient K in formula (5). The agreement between calculated and experimental values in Table 2 indicates that for stirrups inclined at 67.5 degrees, the factor Krf_y is about 20 per cent greater than for stirrups at 45 or 90 deg. This factor Krf_y can be thought of as representing the effect of primary stresses in the truss with rigid connections. The other two factors in formula (5) would represent the effect of secondary stresses in that truss due to the rigidity of the connections. The effect of these secondary stresses decreases somewhat the advantage given by the stirrups inclined at 67.5 deg. over those inclined at 45 or 90 deg. In Fig. 7, the theoretical ratio between the shearing stresses at V_y for stirrups at 67.5 deg. and those for stirrups at 45 and 90 deg. have been represented. The calculations have been made for a beam with one per cent of longitudinal steel and assuming the coefficient s equal to 5000. The value of f_y has been taken as 40,000 psi. With these values, v_y is given by:

For stirrups at 67.5 deg. $v_y = 1.20r \times 40{,}000 + 0.04f'_c + 5000 \times 0.01$

For stirrups at 45 or 90 deg. $v_y = r \times 40{,}000 + 0.04f'_c + 5000 \times 0.01$

From Fig. 7, it can be seen that the efficiency of the stirrups at 67.5 deg. compared to those at 45 or 90 deg. depends on the ratio of web reinforcement as well as on the concrete strength. For a concrete strength of 3000 psi., and varying the ratio of web reinforcement from 0.2 to 1.00 per cent, the ratio represented ranges between 1.064 and 1.14. If these calculations are made for 2 per cent of tension steel, the range of the ratio for the same conditions is from 1.053 to 1.129.

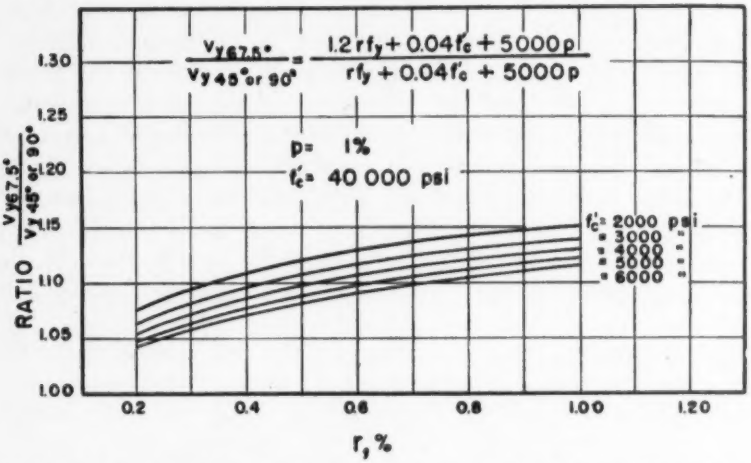

Fig. 7—Ratio between shearing stresses at V_y for stirrups at 67.5 deg. to those for stirrups at 45 deg. or 90 deg. as a function of the ratio of web reinforcement.

The current use of welded stirrups in reinforced concrete beams seems economically possible only if a scheme is worked out so that welded units similar to those shown in the photographs of this paper are prepared in advance in the shop, so that when they arrive at the construction where they are to be placed, they will require a minimum of workmanship and handling. In this case, advantage should be taken of the better efficiency given by stirrups placed at 67.5 deg. to the axis of the beams.

The safe shearing stress that the stirrups will be able to resist should be based on the value of v_y, which for these tests was equal to:

$$v_y = Krf_y + 0.04f'_c + 5000p \text{ (in psi)} \quad \ldots\ldots\ldots\ldots (6)$$

K being 1.00 for stirrups placed at 45 or 90 deg. and 1.20 for stirrups at 67.5 deg. The factor of safety by which v_y should be divided is a matter of judgment. It is believed that in selecting the factor of safety, some extra allowance should be made in the factor $5000p$ due to the unknown effect of the longitudinal bars.

The following formula for the safe shearing stress is proposed:

$$v_s = Krf_s + 0.02f'_c + 2000p \text{ (in psi)} \quad \ldots\ldots\ldots\ldots (7)$$

Fig. 8 gives the factor of safety, $\frac{v_y}{v_s}$, as a function of the ratio of web reinforcement for several concrete strengths. The computations have been made for a beam with one per cent of longitudinal reinforcement and stirrups at 67.5 deg. The value f_y has been assumed to be 40,000 psi and that of f_s to be 20,000 psi.

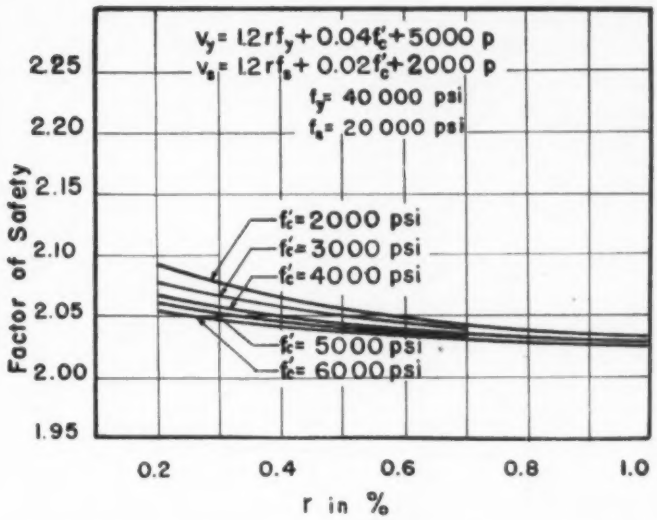

Fig. 8—Factor of safety, with reference to yielding of web reinforcement $\frac{V_y}{V_s}$, as a function of the ratio of web reinforcement for p = 0.01 and stirrups at 67.5 deg.

Due to the fact that the factor Krf is more important for stirrups at 67.5 deg. than for those at 45 and 90 deg., the factor of safety, computed as before, is a little greater for these latter types of stirrups.

Fig. 9 gives the relation between the shearing stresses at ultimate load determined by the formula

$$v_u = Krf_y + 0.10f'_c + 5000p \quad \ldots\ldots\ldots\ldots\ldots\ldots 8)$$

and the safe shearing stresses computed from formula (7). These values, given for various concrete strengths, are plotted as ordinates against abscissas representing the ratio of web reinforcement. In computing these values, a beam with one per cent of longitudinal reinforcement and stirrups at 67.5 deg. was assumed. The value of f_y was taken as 40,000 psi. and that of f_s as 20,000 psi.

This factor of safety depends greatly on both the concrete strength and the ratio of web reinforcement. It is more than 3 for small ratios of web reinforcement and as low as 2.43 if one per cent of web reinforcement and a weak concrete (2000 psi.) is used, the ratio of longitudinal reinforcement being also one per cent.

6—COMPARISON WITH FORMER TESTS ON LOOSE STIRRUPS

A survey was made of former tests on web reinforcement in order to compare the behavior of welded stirrups with the conventional loose stirrups. It was found that practically all of the beams (of sizes comparable to those commonly used in buildings and other structures) that

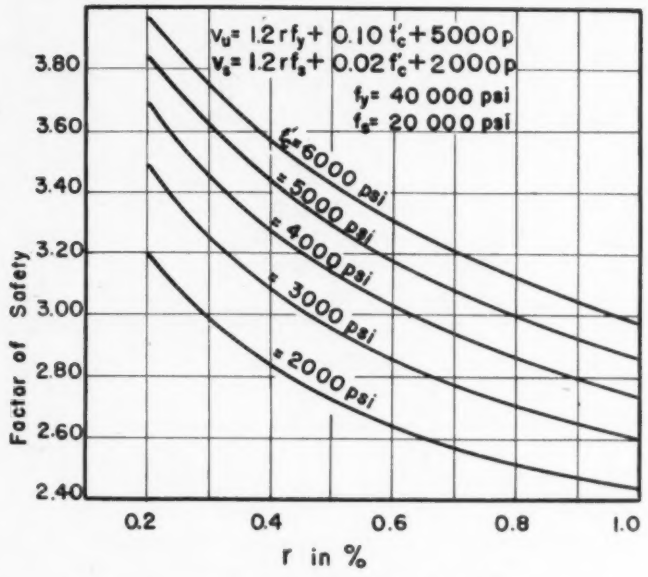

Fig. 9—Factor of safety, based on ultimate strength $\frac{v_u}{v_s}$, as a function of the ratio of web reinforcement for p = 0.01 and stirrups at 67.5 deg.

were previously tested had failed by causes other than diagonal tension. The only tests that could be used for this purpose are those reported by Slater, Lord, and Zipprodt.*

The specimens used in that investigation were short, deep, reinforced concrete I-beams loaded at the center and provided with stiffeners at the support and the center. Because of the heavily reinforced flanges and the presence of vertical stiffeners, the beams resembled Vierendeel girders with a thin web between the main members. Due to this, the action of those beams was probably different from that of the beams reported herein. For this reason, although a comparison has been attempted, it is believed that it does not settle the problem, but gives only a general idea of what might be expected.

Table 3 shows by reference number, as given in the report cited, the specimens selected for this comparison. No specific yield point stress is given for the steel; it is only stated that "the yield point stress of the shrapnel steel bars varied with the size of the bar, with some degree of regularity, from about 55,000 psi. for 1¼ in. to about 70,000 psi. for ⅜-in. bars." The values shown in Table 3 have been determined by assuming a straight line variation between the extreme values noted.

*W. A. Slater, A. R. Lord, and R. R. Zipprodt, "Shear Tests of Reinforced Concrete Beams," Tech. Paper No. 314 of the Bureau of Standards.

TABLE 3—

COMPARISON OF THE EFFICIENCY OF WELDED AND LOOSE STIRRUPS

Based on test values for loose stirrups (Technologic Paper 314, U. S. Bureau of Standards) and values calculated from Equation 8.

Ref. No.	$f'c$ psi.	p, %	r, %	f_u psi.	v_u psi.	v_u corrt'd, psi.	v_u calc.	Ratio $\frac{v_u \text{ calc.}}{v_u}$	Ratio $\frac{v_u \text{ calc.}}{v_u \text{ corrt'd}}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
13	5970	2.3	1.30	63,500	1030	950	1540	1.49	1.62
18	3510	2.3	1.30	63,500	970	900	1290	1.33	1.43
23	5580	1.73	2.60	65,500	1700	1470	2345	1.38	1.59
28	3690	2.33	0.92	65,500	980	910	1085	1.11	1.19
34	5550	2.02	1.55	68,000	1550	1360	1710	1.10	1.25
35	5160	1.41	1.66	68,000	1430	1220	1715	1.20	1.40
38	5750	2.04	1.20	68,000	1310	1150	1495	1.14	1.30
40	5720	2.26	0.83	68,000	1120	1010	1250	1.11	1.24
41	5700	2.34	0.59	68,000	880	805	1090	1.24	1.35
44	4720	2.04	1.77	68,000	1580	1390	1775	1.12	1.28
46	4150	2.29	0.86	68,000	1060	930	1115	1.05	1.20
52	5920	1.98	0.81	70,000	1290	1090	1260	0.98	1.15
53	5170	2.03	0.95	70,000	1410	1200	1285	0.91	1.07
56	4170	2.28	1.16	70,000	880	600	1345	1.53	2.24*
69	5500	2.01	2.38	65,500	1860	1640	2210	1.19	1.35
70	5310	2.04	2.52	65,500	1840	1650	2285	1.24	1.38
71	3650	2.04	2.69	65,500	1960	1820	2225	1.14	1.22
72	5400	2.02	1.58	68,000	1790	1600	1815	1.01	1.13
73	5190	1.99	1.49	68,000	1730	1530	1630	0.94	1.07
74	4380	2.04	1.16	68,000	1220	1075	1330	1.09	1.24
76	5950	2.24	0.83	68,000	1290	1210	1270	0.99	1.05
77	4990	2.34	0.58	68,000	970	910	1010	1.04	1.11
78	5160	2.02	1.67	68,000	1720	1490	1755	1.02	1.18
81	5230	2.02	0.87	70,000	1400	1210	1235	0.88	1.02
82	4380	2.01	0.84	70,000	1300	1120	1125	0.87	1.00
83	3240	2.04	0.94	70,000	1310	1120	1085	0.83	0.97
Average								1.11	1.23

*Not considered in the average.

This study has been limited to a comparison of the probable shearing stresses that the beams would resist at ultimate load were they reinforced with welded stirrups as calculated by formula (8); and the actual ultimate shearing stress given by the tests. Of the 114 beams reported in Tables 7 and 8 of the Technological Paper No. 314, 26 were selected for this purpose. The selection was made by considering only those with web reinforcement similar to that used in this investigation reported to have failed in diagonal tension and in which the stress in the web reinforcement at failure had reached the yield point. Column 6 shows the shearing stresses at ultimate load, based on gross section, as given in Tables 7 and 8. These stresses, reported to have been corrected to allow for the stiffness of the heavy flanges and stiffeners of the test beams, appear, from the maximum loads listed, to be uncorrected. The writer has therefore applied a correction to the shearing stresses listed in Technological

Paper No. 314, determined as indicated on page 414 and based on the maximum deflections listed on page 492 of that paper. Values of the maximum shearing stresses, uncorrected and corrected, are given in Columns 6 and 7 of Table 3.

For comparison, the writer has computed values of v_u by use of Eq. (8). For this purpose, p has been taken as the ratio of the tension steel section to the rectangular section enveloping the I-beam cross section. The ratio of this calculated value to the uncorrected and corrected values of Columns 6 and 7 is shown in Columns 9 and 10 of Table 3. These ratios show considerable spread in the individual values, as might be expected. On the average, it can be said that the strength of welded stirrups may be expected to be between 10 and 25 per cent greater than that of loose stirrups.

Due to the heavy reinforcement provided in the compression flange of the reinforced concrete I-beams it is believed that the values given in Column 10 represent more nearly what could be found on similar specimens with welded and loose stirrups. From the discussion in Section 5 of this paper, it is to be expected that the strength provided by the reinforced concrete I-beams after the load corresponding to V_y had been reached, was comparatively much greater than that given by the specimens reported in this investigation. It seems reasonable to expect the welded stirrups to be, on the average, about 20 per cent stronger than similar loose stirrups. The use of welded stirrups with 67.5 degrees inclination will make these stirrups 25 to 35 per cent stronger than the loose stirrups placed at 45 or 90 deg. Furthermore, while inclined loose stirrups were used in the laboratory tests of Technological Paper No.314, it is not likely that such stirrups would be practical or satisfactory in building construction.

7—SUMMARY AND CONCLUSIONS

This investigation embodies the tests of 44 beams of reinforced concrete of a size comparable to that commonly used in buildings and other structures, and designed in such a way as to produce failure by diagonal tension. The tests were made on simple beams subjected to two-point loading, and the web reinforcement was placed in the region of constant shear. A fairly large amount of tension steel was used to avoid failure by longitudinal tension or bond. The web reinforcement was provided by straight stirrups welded to the main reinforcement to form a unit. Variables including the size and inclination of the stirrups and the type of concrete used were studied. In four of the beams the ratio of main reinforcement was varied, in which case the quantity of tension steel was more nearly comparable to that commonly used in reinforced concrete beams.

The following are general remarks based on the results and discussion of these tests:

(a) The beams without web reinforcement failed by diagonal tension at an average shearing stress of about 490 psi.

(b) For beams with web reinforcement, an analysis was made of the test results, with particular attention to two stages of loading: (1) for the load at which the yield point stress of the web reinforcement was reached, as defined in Section 3, and (2) for the ultimate load. For both cases a formula was developed for the shearing stresses. For the load at which the web reinforcement was stressed to the yield point, the shearing stress is given by:

$$v_y = Krf_y + 0.04f'_c + 5000p \text{ (in psi.)} \quad \ldots\ldots\ldots\ldots (6)$$

At ultimate load, the shearing stress is given by:

$$v_u = Krf_y + 0.10f'_c + 5000p \text{ (in psi.)} \quad \ldots\ldots\ldots\ldots (8)$$

(c) The safe shearing stress that the stirrups will be able to resist should be based on formula (6). The factor of safety by which v_y should be divided is a matter of judgment.

The following formula for the safe shearing stress is proposed:

$$v_s = Krf_s + 0.02f'_c + 2000p \text{ (in psi.)} \quad \ldots\ldots\ldots\ldots (7)$$

This formula gives a factor of safety a little greater than two with respect to v_y and a mimumum of 2.43 with respect to v_u when a 2000 psi. concrete is used in a beam that has one per cent of both web and main reinforcement, with $f_y = 40{,}000$ psi. and $f_s = 20{,}000$ psi.

(d) A study was made of the ratio of the strengths of beams made with welded and loose stirrups. For that purpose, 26 beams were selected from the series of tests reported by Slater, Lord, and Zipprodt in Technological Paper No. 314 of the Bureau of Standards.

Although relatively short beams were used in both series of tests studied, the comparison is rendered uncertain by the fact that the two series of tests were made 25 years apart, at different laboratories, and with different materials, techniques, and types of test beams. From the comparison, it seems reasonable to expect a beam made with welded stirrups to have on the average, about 20 per cent greater resistance to diagonal tension than one containing similar loose stirrups. Furthermore the use of welded stirrups with a 67.5 deg. inclination should make a beam 25 to 35 per cent more resistant than one made with loose stirrups placed at 45 or 90 deg.

(e) A comparison was made between the measured deflections at midspan and the theoretical deflections due to flexure, as calculated by Maney's equation. The measured deflections at ultimate load were sometimes as much as 40 per cent greater than the calculated ones. At

the load corresponding by V_y, they were, on the average, 10 to 20 per cent greater than the calculated values. This indicates the presence of shearing deflections and deflections due to opening of diagonal cracks of appreciable importance in these test beams.

ACKNOWLEDGMENT

These tests were made in the University of Illinois Engineering Experiment Station under the supervision of Prof. Frank E. Richart. The writer is very much indebted to Professor Richart for many valuable suggestions and criticisms throughout this investigation.

ACI NEWS LETTER

Vol. 17 No. 2 JOURNAL of the AMERICAN CONCRETE INSTITUTE November 1945

42nd ANNUAL ACI CONVENTION
New York City—Feb. 18-21, 1946

The times have not yet come when hotel accommodations are "easy". Make your plans early to attend the ACI meetings in New York City and MAKE YOUR RESERVATIONS EARLY. The management of Hotel New Yorker has promised to take care of all ACI people—but that promise is based on your responsibility to make early and specific reservations. In making these reservations be sure to identify yourself with ACI.

Timely and varied subject matter will engage six general sessions of the Institute's 42nd Annual Convention at Hotel New Yorker, New York City, Tuesday, Wednesday and Thursday, February 19 to 21, 1946.

This will be ACI's first postwar get-together and the first in the nation's metropolis since 1939—not counting the pocket-size, purely administrative, meetings last February.

Some outstanding objectives in subject matter were discussed by the Board of Direction in September and referred to the Publications Committee, Robert F. Blanks, Chairman. The Committee is at work to produce a good program. To all ACI Members it might be said: If you are asked to help, try to do it. If you have an idea—a suggestion of a worth-while source for a JOURNAL paper likely to be of interest to many readers, pass it on to Mr. Blanks (Bureau of Reclamation, Denver) with a copy to the ACI Secretary. It is well to remember that not all good papers are good convention papers. The rostrum presents problems different from the printed page.

"Leaders" have been named, each of whom is to develop a subject coverage—not in one paper but in several brief, brisk convention presentations which will leave laborious detail for study in the printed page, from which such de-

tail can be more readily absorbed. Papers offered for each session are first to be prepared in full in a version intended for publication—then be briefed by their authors for convention purposes.

Board of Direction and Advisory Committee meetings are scheduled to be held Monday, February 18, with meetings that day also of such technical committees as do not involve personnel conflicts with Board or Advisory Committee. Other technical committees are planning to meet Tuesday morning, February 19.

General convention registration will begin at 9 a.m. on the 19th. The following sessions are scheduled:

Tuesday, February 19 —2 p.m.
8 p.m.
Wednesday, February 20—9.30 a.m.
2.00 p.m.
8 p.m.
Thursday, February 21 —9.30 a.m.

Subjects scheduled tentatively for special attention are:

Reinforced concrete design theory and practice.

Concrete construction practice.

Research (under the customary auspices of ACI Committee 115, Morton O. Withey, Chairman, and S. J. Chamberlin, Secretary).

Maintenance and repair of concrete structures.

Use of air-entraining concrete.

In addition to these, a sprinkling of diversified papers.

WHO'S WHO in this JOURNAL

Glenn S. Paxson

ACI member since 1930, is Bridge Engineer for the Oregon State Highway Commission. He received his B.S. degree from Oregon State College in 1912 and a C.E. degree in 1937 from the same school. Following his graduation he was for two years assayer and surveyor for the Ben Harrison Mines Co. in Eastern Oregon, and then for three years was superintendent of quarry operations producing surfacing material for highway construction. He served as 1st Lieut. during the First World War with eighteen months' service in France. Upon returning to civil life, he entered the employ of the Oregon State Highway Commission and has remained continuously with this organization successively as transitman, resident bridge engineer, assistant bridge engineer in charge of con-

struction, and bridge engineer, serving in the latter position since 1935. See his contribution on maintenance and repair of concrete bridges p. 105.

T. C. Powers

ACI member since 1927—twice a Wason Research Medalist, is too well known as a contributor to this JOURNAL and as Manager of Basic Research, Portland Cement Association, Chicago, to need an introduction in this column on the occasion of his paper "Should Portland Cement Be Dispersed?" p. 117.

Oreste Moretto

ACI member only since 1944, is a graduate of the University of Litoral, Rosario, Argentina, where he received the degree of Civil Engineer in 1940. From 1940 to 1943 he was engaged on bridge design for the Public Roads Administration of Argentina. Since September, 1943, he has pursued graduate studies in Civil Engineering at the University of Illinois, working under fellowships from the University of Litoral and the Institute of International Education in New York City.

Mr. Moretto received the degree of M.S. in Civil Engineering in 1944 and is now completing his work for the Ph. D. degree. He expects to return to his native country, Argentina, in 1946. His contribution to this JOURNAL p. 141 grew out of work with Prof. F. E. Richart at Talbot Laboratory, University of Illinois.

Phaon H. Bates Retires

Phaon H. Bates, chief of the Division of Clay and Silicate Products, National Bureau of Standards, and a member of the Bureau's staff since 1910, exercised his optional retirement privilege Sept. 15, 1945. Mr. Bates was born August 1, 1879, in Sipisville, Somerset County, Pa. and received his formal education in the public schools of Philadelphia and at the University of Pennsylvania, choosing chemistry as his profession. He served in the chemical laboratory of the Pennsylvania Railroad where he analyzed miscellaneous supplies and later joined the staff of their locomotive testing plant where he worked on coals, ashes, and combustion products. From 1906 to 1910 he served as a chemist in the Pittsburgh laboratory of the U. S. Geological Survey.

When on July 1, 1910, the functions of that laboratory were divided between the newly established Bureau of Mines and the National Bureau of Standards, he was transferred to the latter. Continued as the Pittsburgh branch of the Bureau, a laboratory was maintained on the grounds of the old U. S. Arsenal in that city for ceramic research and the testing of structural materials; he became principal chemist of this laboratory, and was placed in entire charge of the work. Because of unsettled conditions prior to World War I, it appeared that the United States would be cut off from its supply of optical glass, and facilities for its production were not available in this country. Mr. Bates organized an experimental program that ultimately led to the manufacture of high grade optical glass in the Pittsburgh laboratory.

On the completion of the Bureau's Industrial Building in 1919, the major portion of the work of the Pittsburgh branch was transferred to Washington where it was consolidated with other projects to form a new division concerned with engineering and structural materials. This eventually became the present Division of Clay and Silicate Products, covering the fields of whiteware, optical glass, refractories, ceramic coatings, cement and concrete, lime and gypsum, stone, and masonry construction. This division played a leading part in the extensive test program on building materials and structures conducted during the years just preceding the war, the results of which have been published in a series of more than one hundred reports.

During World War II, the optical glass laboratory assumed the proportions of a fair-sized industrial plant with all the usual difficulties of recruiting personnel and meeting exacting production schedules. That these difficulties were overcome is due in no small measure to the work of Mr. Bates. Another important contribu-

tion of the Division was the development of a new ceramic coating for the protection of metal parts against corrosion at high temperatures. This was used on the exhaust system of aircraft and amphibious vehicles.

Mr. Bates has always taken a leading part in the development of testing methods and likewise in the preparation of Federal and other nationally-used specifications. In recognition of his outstanding reputation in this field, he was elected president of the American Society for Testing Materials at the 47th annual meeting in June 1944, serving until June 30, 1945. He delivered the 1940 Edgar Marburg Lecture to this Society, a lecture given annually by an outstanding technical authority as a means of emphasizing the promotion of knowledge of engineering materials.

He has been closely identified for many years with the work of the American Ceramic Society of which he is a fellow, and of the American Concrete Institute. President of the Institute from 1934 to 1936, he was awarded its Turner Gold Medal in 1939 "for contributions to science, direction of research and outstanding leadership in advancing the intelligent utilization of cement and concrete." A member of ACI continuously from 1926 and elected a director in the same year, he had been an active member in the Institute's earlier years. He had been member and chairman of its Publications Committee, member of Advisory Committee, member and chairman of its former Program Committee. He was also a member of the ACI Standards Committee, from which he resigned at the time of his retirement. He had always been an active advocate of increased work on the development of standards. He is a fellow of the American Association for the Advancement of Science and a member of the American Chemical Society, the Washington Academy of Sciences, and the Cosmos Club.

Author or co-author of several Bureau papers dealing with portland cement and related subjects, he has also contributed numerous articles to the technical journals in his special fields of activity.

Douglas E. Parsons has been named chief of the Division of Clay and Silicate Products to succeed Mr. Bates.

Douglas E. Parsons

President of American Concrete Institute, who has been a member of the staff of the National Bureau of Standards for more than twenty years, became, in September, Chief of the Bureau's Division of Clay and Silicate Products, to succeed Phaon H. Bates, who retired from Government service on Sept. 15, 1945. Mr. Parsons, a civil engineer, has been associated with the Bureau's section on masonry construction.

A native of Iowa, born in Marion, Nov. 3, 1894, Mr. Parsons received his formal education in his home state, graduating in 1917 with the A.B. degree and receiving the C.E. degree in 1923, both from Cornell College, Mount Vernon, Ia. In the United States Army from 1917-19, he served as a Lieutenant with the 89th Division, American Expeditionary Forces. Upon his return to civilian life, he became Assistant Chief Engineer of the Iowa Railway and Light Co., Cedar Rapids.

In 1923 he joined the staff of the National Bureau of Standards, was placed in charge of the Masonry Construction Section in 1929, and early in 1944 was designated assistant chief of Division IX which he now heads; his entire service with the Bureau has been in that division.

Following many years of active participation in the affairs of the American Concrete Institute, Mr. Parsons was elected its President in February of this year, for a one-year term. In 1936, he was awarded the Institute's Wason Medal in recognition of noteworthy research; he shared this honor jointly with Ambrose H. Stang of the National Bureau of Standards and Vice-Admiral Ben Moreell, U. S. Navy, Bureau of Yards and Docks, for work involved in their studies of Mesnager Hinges used in concrete construction. Through a coincidence, the medal was presented to him by P. H. Bates, then President of the Institute, whom he has now succeeded as division chief.

Mr. Parsons has been an Institute member since 1926, a member of its Board of Direction since 1941 and served as member of the Advisory Committee and of its Publications Committee, having been chairman of the latter 1941 to 1945.

He is a member of the American Society of Civil Engineers, the American Society for Testing Materials, the American Association for the Advancement of Science, the Society for Experimental Stress Analysis, and the Washington Academy of Sciences; and is Chairman of American Standards Association Committee A-41 of Masonry, and the Federal Specifications Technical Committee on Precast Reinforced Concrete Products. Mr. Parsons is author or co-author of a large number of papers in the Bureau's series of publications, including more than a dozen of the Building Materials and Structures Reports on structural properties of various types of masonry construction. He has also contributed numerous articles and papers to technical journals in his field.

New Members

The Board of Direction approved 47 applications for Membership (35 Individual, 7 Corporation, 2 Junior, 3 Student) received in August and September as follows:

Billig, K., 167 Victoria St., London, S. W. 1, England

Bohn, Richard A., Nat'l Bureau of Standards, 209 Old Mint Bldg., San Francisco 3, Calif.

British Reinforced Concrete Engineering Co. Ltd., Stafford, England (C. H. Griffin)

Benavant y Campama, Jaime P., Edificia Banco, Nova Scotia 224, Havana, Cuba

Carroll, P. J., "Wave Crest", Ennis Crone, 60 Sligo, Eire

Chambers, J. W., Alabama State Highway Dept., Montgomery, Ala.

Cohen, George N., c/o Euclid Construction Co., 101 Park Ave., New York 17, N. Y.

Diaz, Agapito Leon, Princesa 119 Jesus del Monte, Havana, Cuba

Forsman, Otto, Statens Provningsanstalt, Stockholm 26, Sweden

Funk, Charles R., 6a Avenida Sur, Numero 54, Guatemala City, Guatemala, C. A.

Guillo, Carlos Maruri y, 14 entre 3a y 5a, Miramar, Havana, Cuba

Habashi, Fauzi, Faculty of Engineering, Fouad 1 st University, Giza, Egypt

Hatch, George E., c/o U. S. Engineer Office, Honolulu, T. H.

Hawaiian Cement Co. Ltd., P. O. Box 2454, Honolulu, Hawaii (L. N. Bryant)

Hebold, Denis, 217 W. Sparks St., Philadelphia 20, Pa.

Hong, Li Ching, c/o The Librarian, Bureau of Reclamation, Denver 2, Colo.

Huckleberry, Major B. C., New Belvedere Hotel, Columbus, Indiana

Highway State Comm. of Kansas, Masonic Temple Bldg., Topeka, Kansas (R. D. Finney)

Kendall, Edgar R., 2360 Glenwood Ave., Toledo, Ohio

Krueger, Arthur, Box 77, R. R. 3, Cincinnati 11, Ohio

Long, J. S., Devoe & Raynolds Co., Inc., P. O. Box 328, Louisville, Ky.

Loomis, Robert S., A/S, Navy V-12 Unit, M.I.T., Room No. 518-A, Cambridge 30, Mass.

Matthews, Homer M., Matthews & Kenan, 1616 Transit Tower, San Antonio, Texas

Mautner, Dr. K. W., 65 Ladbroke Grove W., 11 Flat 3, London, England

Painter, William D., 101 Lincoln Street, Alcoa, Tenn.

Pallot, D. P., 123 Derby Rd., Subiaco, West Australia

Pang, Dat Quon, 938 N. Vineyard St., Honolulu 7, T. H.

Panuzio, Frank L., 1385 Capitol Ave., Bridgeport, Conn.

Parva Domus, S. A., Apartado 1095, Lima, Peru, S. A.

Pennsylvania-Dixie Cement Corp., Old Colony Bldg., Des Moines 9, Iowa

Pittard, A. R., 39 Victoria St., London, S. W. 1, England

Polychrone, James A., Nichols 102, M.I.T. Dorms, Cambridge, Mass.

Roberts, C. G., Country Roads Board, Carlton, N3, Victoria, Australia

Roche, Walter W. R., 6 Haarlem Road, London W. 14, England

Rosberg, E. O., 1490 Marin Ave., Albany, Calif.

Schuetz, Clyde C., United States Gypsum Co., 1253 Diversey Parkway, Chicago 14, Ill.

Sementtiyhdistys, Kalevankatu 20, Helsinki, Finland

Shevling, Charles E., Ross Dam, Rockport, Wash.

Shoemaker, Robert R., P. O. Box 318, Long Beach, Calif.

Spindel, M., 89 Dorchester Court, Flerne Hill, London, S. E. 24, England

U. S. Engineer Office, 409 Davidson Bldg., 10 East 17th St., Kansas City 8, Mo.

Upper, R. E., 53 Fulton Ave., Toronto, Ont., Canada

Wagenett, Frank, 28 Mountain St., Rockville, Conn.

Wai, Francis K., 3425 Pahoa Ave., Honolulu, Hawaii

Waidelich, A. T., The Austin Co., 16112 Euclid Ave., Cleveland 12, Ohio

Wessels, Vincent E., Petoskey Portland Cement Co., Petoskey, Mich.

Wright, K. W., Ash Grove Lime & Portland Cement Co., P. O. Box 519, Chanute, Kansas

Honor Roll

February 1 through October 31, 1945

Rene Pulido y Morales, in Havana, Cuba heads the list with 21 new Members proposed since Feb. 1.

Rene Pulido y Morales	21
Roy Zipprodt	6
Harry B. Dickens	5
H. F. Gonnerman	4
A. Amirikian	3
J. H. Spilkin	3
Charles S. Whitney	3
Ernst Gruenwald	2½
Charles E. Wuerpel	2½
C. Blaschitz	2
H. W. Cormack	2
Francis MacLeay	2
Calvin C. Oleson	2
D. E. Parsons	2
Dean Peabody, Jr.	2
C. H. Scholer	2
J. M. Wells	2
J. W. Kelly	1½
Ben E. Nutter	1½
O. A. Aisher	1
H. Victor Carman	1
W. Fisher Cassie	1
A. R. Collins	1
R. F. Dierking	1
H. F. Faulkner	1
P. J. Freeman	1
J. K. Gannett	1
Stanley S. Haendel	1
G. H. Hodgson	1
F. B. Hornibrook	1
V. P. Jensen	1
L. I. Johnstone	1
William G. McFarland	1
Denis Matthews	1
H. W. Mundt	1
Y. G. Patel	1
A. F. Penny	1
Kenneth Powers	1
Simeon Ross	1
John A. Ruhling	1
Byram Steel	1
G. W. Stokes	1
Wm. Summers Jr.	1
M. A. Timlin	1

J. W. Tinkler.....1
Maxwell Upson.....1
Stanton Walker.....1
J. C. Witt.....1

The following credits are, in each instance, "50-50" with another Member.

Birger Arneberg	Adolph Meyer
Michel Bakhoun	A. F. Moore
E. E. Bauer	O. F. Moore
E. W. Bauman	M. D. Olver
P. G. Bowie	C. E. O'Rourke
C. H. Chubb	Jerome Raphael
J. H. Chubb	F. E. Richart
Arthur P. Clark	Chas. S. Rippon
R. R. Coghlan	Kanwar Sain
R. B. Crepps	J. L. Savage
Harmer E. Davis	Oskar Schreier
N. M. Hadley	A. L. Strong
W. C. Hanna	A. C. Trice
W. S. Hanna	J. H. de W. Waller
Carl W. Hunt	A. R. Waters
W. R. Johnson	David Watstein
Paul A. Jones	George Winter
H. J. McGillivray	Harry C. Witter
R. E. McLaughlin	K. B. Woods

Frank W. Garran

dean of the Thayer School of Engineering at Dartmouth College, ACI member since 1942, died Sept. 19 in the college infirmary at the age of 51. He was stricken with nephritis four months ago. He was born in Boston, 1894.

Dean Garran, associated with Thayer School since 1929, had been dean since 1934. Recently he was chairman of the New England section of the Society for the Promotion of Engineering Education and at his death was a national director of the society representing New England. He was also a trustee of Norwich University.

A graduate of Norwich in 1917, he was headmaster of the Atkinson Academy in New Hampshire for a year before returning to the university as Assistant Professor of Civil Engineering 1920 to 1923. He took his master's degree at Massachusetts Institute of Technology in 1924 and in that year became assistant professor of civil engineering at the University of Arizona. Two years later he was named Professor of Engineering at the College of Charleston, S. C., leaving there in 1929 to come to Dartmouth as assistant professor. Dartmouth awarded the honorary master's degree to him in 1933, the year in which he was made full professor and acting dean of the Thayer School.

During the war he was coordinator of Dartmouth's civil pilot training program and also directed the Government-sponsored engineering, science, management war training course. In the first World War he was a first lieutenant with the Army engineers and served in France and Germany. He was a former president of the Dartmouth Scientific Association, a member also of the American Society of Civil Engineers and Theta Chi Fraternity.

George T. Seabury

who died May 25, 1945, on the eve of his retirement as Secretary of the American Society of Civil Engineers, after 20 years service with the society, had been an ACI member since 1926.

Mr. Seabury went to the A.S.C.E. in 1925 with a background of practical experience. Born in Newport, R. I., in 1880 and graduated from M. I. T. in 1902 as a civil engineer, he spent four years as a contractor's engineer around New York. The next 9 years were spent on Catskill work of the New York Board of Water Supply.

In 1915 Mr. Seabury went to Providence, R. I. to become a division engineer with the water supply board at that city. Three years later he left that position to accept a commission as a major in the Construction Division of the Army, participating in the construction of Camps Devens, Upton, Meade, Mills, Merritt, Dix and Lee. In 1918 he organized the contracting firm of George T. Seabury, Inc., and was engaged in construction work until 1923 when he was named manager of the Providence Safety Council. He left that position to become secretary of the A. S. C. E. on Jan. 1, 1925.

Guy O. Gardner

ACI member since 1930, who was production manager and research director of all the Ash Grove Lime & Portland Cement plants in Kansas, Missouri, and Nebraska, died August 5 at his home in Chanute,

Kan., age 57. He had been ill with a heart ailment.

Mr. Gardner served as superintendent of the Chanute, Kan. plant from 1925 until 1945 when he received the promotion to production manager and research director. Coming to the Chanute plant June 25, 1912, as a chemist, he held that position until 1917, when he was promoted to assistant superintendent,

He was an active member of the Rotary club, and served as president and also as secretary. He was a past president of the Chanute Country Club and was serving as a director at the time of his death. He was also a director of the Chamber of Commerce, and a member of the Masonic lodge, the Elks, and Presbyterian church.

PCA regional office changes

have been effective since June 15, as announced by William M. Kinney, General Manager of the Portland Cement Association.

M. J. McMillan, Manager of the Washington Office since 1936, is now Regional Manager of the Eastern Offices, 347 Madison Ave., New York 17.

James E. Dunn, District Engineer of the Richmond, Va. office since 1938, is now Manager of the Washington Office, 837 National Press Building, Washington 4, D.C.

Gordon S. Maynard, field engineer in North Carolina and Virginia for the association since 1937, is now District Engineer with headquarters at 1210 State Planters Bank Building, Richmond 19, Va.

Working with Mr. McMillan in supervising association activities in the eastern region is E. M. Fleming, District Manager, New York, in charge of field activities in Connecticut, Maine, Massachusetts, New Hampshire, New Jersey, New York, Rhode Island and Vermont, and G. C. Britton, District Manager, Philadelphia, covering Delaware, Maryland and Pennsylvania.

Messrs. McMillan, Dunn, Fleming and Britton are all ACI members.

Lt. Col. E. W. Scripture

recently returned from Europe after serving three years with the Army Engineers, resumed his post October 8 as Director of Research of The Master Builders Co., Cleveland, O.

Colonel Scripture entered the service in 1942 as a captain, and was assigned to the Office of the Chief Engineer, European Theater of Operations. The first two years of his service overseas were in England where he took part in planning the invasion. In August 1944 Colonel Scripture went to France where he continued planning for engineering operations on the continent. He was awarded the Purple Heart, Bronze Star Medal, Legion of Merit and presented with the Medaille, Louis de Broglie, by the Association des Engeineurs Docteurs de France.

Colonel Scripture, also a veteran of World War I, having served in France for two years with the 26th Division, is widely known in the construction industry for his research work in the improvement of concrete and mortars.

C. D. Williams

member of the Institute since 1937, who has been head of the Department of Structural Engineering at Fenn College, Cleveland, since 1937, except for leave of absence in the war years,accepted appointment as head of the Civil Engineering Department at the University of Florida, effective September 1, Gainesville, Fla. While recently on leave from Fenn College, Professor Williams was supervisor of stress engineers at the Fisher-Cleveland Aircraft Division in charge of stress work on the B-29 and still later as assistant supervising engineer for the J. E. Greiner Co., Baltimore, Md.

The Laclede Steel Company, St. Louis, Mo., contributing member of ACI, announces the election of William M. Akin as president and treasurer of the company on September 14, 1945.

ACI publications in large current demand

ACI Standards—1945

148 pages, 6x9 reprinting ACI current standards: Building Regulations for Reinforced Concrete (ACI 318-41); three recommended practices: Use of Metal Supports for Reinforcement (ACI-319-42); Measuring, Mixing and Placing Concrete (ACI 614-42); Design of Concrete Mixes (ACI 613-44); and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—all between two covers, $1.50 per copy—to ACI Members, $1.00.

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer," and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944. $1.25 per copy; 75 cents to Members.

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This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940; 140 pages. Price $1.50—to ACI members $1.00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

Concrete Primer (Feb. 1928)

Prepared for ACI by F. R. McMillan, it had five separate printings by the Institute alone (totalling nearly 70,000 copies). By special arrangement it has been translated and published abroad in many different languages. It is still going strong. In the foreword the author said "This primer is an attempt to develop in simple terms the principles governing concrete mixtures and to show how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete." 46 pages, 25 cents (cheaper in quantity).

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* * *

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Before me, a Notary Public in and for the State and county aforesaid, personally appeared Harvey Whipple, who, having been duly sworn according to law, deposes and says that he is the Editor of the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE and that the following is, to the best of his knowledge and belief, a true statement of the ownership, management (and if a daily paper, the circulation), etc., of the aforesaid publication for the date shown in the above caption, required by the Act of August 24, 1912, as amended by the Act of March 3, 1933, embodied in Section 537, Postal Laws and Regulations, printed on the reverse of this form, to wit:

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A part of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE Vol. 42

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Vol. 18 No. 4 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN Part 2 Dec. 1946

Discussion of a paper by Oreste Moretto:

An Investigation of the Strength of Welded Stirrups in Reinforced Concrete Beams*

By EDUARDO ARNAL†

I have found very interesting the paper and tests of Oreste Moretto which have a great value in confirming the current practice of design established by the Joint Code, since the formula (7) of the paper:

$$v_s = Krf_s + 0.02\,f'_c + 2000\,p \text{ (in psi.)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

is equivalent to the recommended formula of the Joint Code for inclined bars or stirrups (α between 45 and 90 deg.):

$$Avf_s = (v_s - v_c)\,bs\,\frac{1}{sin\,\alpha + cos\,\alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (I)$$

save a small factor, 2000 p, which can be due to the added strength of the welded stirrups and the influence of the main reinforcement and whose value is still uncertain according to Mr. Moretto.

Demonstration:

If we substitute $K = (sin\,\alpha + cos\,\alpha)\,sin\,\alpha$
and

$$r = \frac{A_v}{ab}$$

in the formula (7) we have:

$$v_s = 0.02\,f'_c + sin\,\alpha\,(sin\,\alpha + cos\,\alpha)\,\frac{A_v f_s}{ab} + 2000\,p \quad \ldots\ldots\ldots (II)$$

The formula (I) of the Joint Code can be written:

$$(v_s - v_c) = (sin\,\alpha + cos\,\alpha)\,\frac{A_v f_s}{sb}$$

*ACI JOURNAL, Nov. 1945; V. 42, p. 141.
†Professor of Advanced Design in Reinforced Concrete, Universidad Central de Venezuela, Caracas.

or:

$$v_s = v_c + (sin\ \alpha + cos\ \alpha)\frac{A_v f_s}{sb} \quad \text{(III)}$$

The spacing of stirrups along the axis of the beam s is equal to $\frac{a}{sin\ \alpha}$

and the Joint Code recommends to use $v_c = 0.02 f'_c$, so we have, replacing these values in equation (III):

$$v_s = 0.02 f'_c + sin\ \alpha\ (sin\ \alpha + cos\ \alpha)\ \frac{A_v f_s}{ab} \quad \text{(IV)}$$

which differs only in the term 2000 p of the equation (II). Therefore, when we compare the Joint Code formula (IV) with the experimentally deduced formula (7) of Mr. Moretto we observe that he found for v_s a value slightly higher (20 to 30 psi in normal beams) than the value recommended by the Joint Code, difference which can be easily explained if we recall the higher resistance developed by the welded stirrups and the effect of the main reinforcement in the ultimate resistance of the beam.

So, until more data is collected and the importance of these two late factors is clearly determined, it seemed to me a better policy to follow the recommendations of the Joint Code whose values are on the side of safety.

Separate prints of this title (42-8) and of its sequel next issue will be available from ACI—after the publication of part 2 in February—at 60 cents each—quantity quotations on request.
Discussion or this paper (copies in triplicate) should reach the Institute not later than May 1, 1946.

Title 42-8 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

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Vol. 17 No. 3 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **January 1946**

Shrinkage Stresses in Concrete*

By GERALD PICKETT†
Member American Concrete Institute

SYNOPSIS

Theoretical expressions for deformations of concrete beams and slabs that occur during the course of drying and expressions for distribution of the accompanying shrinkage stresses are derived in Part 1. These expressions are derived on the assumption that the laws governing the development of shrinkage stresses in concrete during drying are analogous to those governing the development of thermal stresses in an ideal body during cooling. Three cases are considered:

(a) slab or beam drying from one face only;
(b) slab or beam drying from two opposite faces; and
(c) prism drying from four faces.

The applicability of the equations to concrete is considered in Part 2 (to appear ACI JOURNAL, February 1946). It is shown that the course of shortening of prisms is in very good agreement with the theoretical equations and that from a test on one prism the shortening versus period of drying of other prisms of the same material differing in size and number of sides exposed to drying can be predicted with fair accuracy if the differences in size are not too great. However, it is shown that the theory must be modified to take into account inelastic deformation and to permit the supposed constants of the material to vary with moisture content and size of specimen if the theory is to be in agreement with all results on all types of specimen of a given concrete.

Various tests are described which, when used in conjunction with the theory, provide a means for studying some of the more fundamental properties of concrete and for predicting the performance of concrete under some conditions in the field.

INTRODUCTION

Concrete, like many other materials, gains or loses water with changes in ambient conditions. With each change in water content the concrete

*Received by the Institute, April 30, 1945.
†Professor of Applied Mechanics, Kansas State College, Manhattan, Kans.; formerly Portland Cement Association Research Laboratory, Chicago.

tends to shrink or swell. As a result of these changes in volume, stresses are produced that may affect the performance of the concrete structure concerned.

In the design of concrete structures some consideration is usually given to the possibility of subsequent shrinkage or swelling, but the computations for stresses usually include only the stresses produced by loads. The computations are made by means of formulas taken from the science of mechanics of materials and the computed stresses so obtained are compared with allowable stresses, considering the type of structure and location of the concrete in the structure. The basic assumptions for the formulas are that the material is homogeneous, isotropic, free from self-strain and obeys Hooke's law.

Concrete is not homogeneous; by nature it is heterogeneous, even including the binding medium itself, hardened cement paste. It is not isotropic. Factors such as sedimentation before hardening tend to destroy what isotropy there might have been. It does not obey Hooke's law except perhaps under instantaneous strains. It is not free from self-strain at any time; the hardened paste may be shrinking while the aggregate may be resisting a change in volume; the regions near the surface may be fairly dry and tending to shrink whereas those farther inward may be much wetter and tending to resist a reduction in volume. In addition, concrete may change with time, becoming stronger and more rigid if conditions are such as to promote additional hydration.

Not only is concrete a much different material from that assumed in the derivations of elementary formulas but the conditions of loading, the tendency toward redundancy (statically indeterminate), and the structural shapes of concrete members are often such as to make the elementary formulas only rough approximations compared with what the same formulas would be for the usual conditions of loading and structural shapes of steel structures which are usually less redundant.

It is not to be inferred that concrete would necessarily be a better material were all of its properties like those assumed by the design formulas. On the contrary, its ability to relieve stress by creep or plastic flow, for example, partly compensates for its inherently low tensile strength and for uncertainties arising from redundancy. To be remembered also is the fact that, in spite of the deficiencies of design-formulas, concrete structures on the whole perform their intended function. Nevertheless, it should be evident that concrete cannot be used as intelligently as it might be and cannot be studied effectively without a better knowledge as to the magnitude and distribution of stresses within it.

The purpose of this paper may be stated as follows:

First, to derive—on the basis of simplifying assumptions in regard to the properties of concrete—expressions for: (a) deformations of, and (b)

the distribution of shrinkage stresses in, concrete beams and slabs during the course of drying.

Second, to show by means of data from specimens under controlled conditions the manner and degree to which the equations apply to concrete.

Third, to suggest methods for studying some of the more fundamental properties of drying concrete.

No attempt will be made here to give a complete analysis of stresses in concrete. In particular, the effect of aggregate particles on the stresses within the hardened paste will not be considered.

Before expressions for shrinkage stresses in concrete can be derived, assumptions must be made in regard to the relation between shrinkage and moisture content and the laws controlling the flow of moisture in concrete as well as the relation between stress and strain.

The actual relationships are not as simple as could be desired. If the flow of water were entirely by vapor diffusion, if the vapor pressure of the water in the concrete were proportional to the moisture-content, and if permeability were independent of the moisture-content, then the differential equation for the flow of water would be a partial-differential equation known in physics and mathematics either as the *diffusion equation* or as the *equation of heat conduction*. Carlson,[1]* in a study of distribution of moisture in concrete, assumed that this equation applies. If the flow of water could be expressed by the diffusion equation and if the shrinkage (or swelling) tendency† of each elemental volume were linearly related to the moisture-content, the unrestrained shrinkage (or swelling) could also be expressed by the diffusion equation. This possibility was also considered by Carlson.[1] But the flow of water is different from that indicated by the diffusion equation, and the relationship between the change in moisture-content and unrestrained shrinkage is not linear as required by these equations. Moreover, satisfactory expressions for either the flow of water or the moisture-shrinkage relation have not been found.

It is believed that moisture in concrete flows partly as liquid in capillaries, partly as vapor, and partly as adsorbed liquid on the surface of the colloidal products of hydration. While drying progresses, the vapor pressure of the water remaining in the region losing water decreases progressively with the moisture content. This change in vapor pressure with change in moisture content is not linear with respect to moisture

*See references at end of text of Part 1.

†By shrinkage (or swelling) tendency is meant the unit linear deformation due to any cause other than stress that would occur in an infinitesimal element if the element were unrestrained by neighboring elements. It is not to be confused with the *average* unit deformation, commonly called shrinkage, of a so-called unrestrained specimen, nor with the resultant linear unit deformation which for the x-direction will be designated e_x. Hereinafter, the linear unit shrinkage tendency will be referred to either as unrestrained shrinkage, for clarity, or merely as shrinkage, for brevity.

content. Neither is the rate of flow proportional to the gradient in vapor pressure. The shapes and relative proportions of the spaces occupied by liquid and by vapor change as drying proceeds. This fact, as well as the non-uniformity of the spaces, is believed to be partly responsible for the way in which vapor pressure depends on moisture content and the way in which rate of flow depends on gradient of vapor pressure.

The volume-change-vs.-weight-loss relation is different for different concretes depending on the composition of the concrete and the conditions of curing. For the same concrete it is different during first shrinkage from what it is during the second or subsequent volume changes. If a saturated prism of concrete is allowed to dry, the ratio of change of length to loss of water increases as drying proceeds. At first, comparatively small changes of volume occur per unit loss of weight. The higher the water-cement ratio and the shorter the period of curing the smaller the change during the initial stages of drying. Later the ratio becomes much larger and remains almost constant for some time, after which it may either increase or decrease as the specimen approaches its final weight. It is believed that this ratio, at any stage of drying, depends upon (a) the shape, size, and degree of uniformity of the spaces that hold the water; (b) the shape, size, rigidity, and spacing of the solid particles; and (c) the strength of the bonds between particles.

The relations between stress and strain must be considered in any study of volume changes resulting from moisture changes in concrete because any tendency for a change in volume that progresses from the surface inward always develops stresses. The stresses in turn, through the stress-strain relation, modify the resultant deformations. For low stresses both the elastic and inelastic strain produced by stress are approximately proportional to the stress, permitting Hooke's law to be assumed, but under most conditions of drying the shrinkage stresses, either alone or in combination with stresses from other sources, may be large enough to cause cracks and structural damage within the concrete and for such stresses the proportionality does not hold. Moreover, the apparent plasticity of an element* is greater during the time the element is drying for the first time than at any other time. The relative positions of the colloidal gel particles are no doubt changed by drying and while the changes are taking place small resultant stresses on an element will produce relatively large inelastic deformations.

In spite of the apparent difficulties of obtaining a satisfactory solution to the problem of deformations and stresses in concrete exposed to changes in ambient conditions, a relatively simple procedure has proved to be rather successful. The procedure is to assume as Carlson did that

*The reference is to an element of hydrated paste of just sufficient size to be a representative sample of the paste.

the diffusion equation applies to shrinkage even though the simple relations that are implied by that assumption are contrary to fact. It is further assumed that concrete follows Hooke's law. The derivations given in Part 1 are based upon these assumptions.

Since in Part 1 the derivations for deformations and stresses are based on the assumptions that shrinkage follows the diffusion equation and the material follows Hooke's law, the equations are even more applicable to thermal stresses in metals than to shrinkage stresses in concrete. In fact, much of the mathematical work given here was taken from the literature on diffusion of heat and on thermal stresses, as the references will show. However, certain corresponding coefficients in the two problems are of an entirely different order of magnitude. For example, the numerical value of the thermal diffusivity for steel expressed in square inches *per second* is approximately the same as the numerical value of the shrinkage diffusivity of concrete expressed in square inches *per day*. Because of the relatively slow diffusion of shrinkage the application of the hypothesis to the shrinkage of concrete necessitates the study of early transient conditions (usually ignored in the treatment of heat).

PART 1—SHRINKING (OR SWELLING), ITS EFFECT UPON DISPLACEMENTS AND STRESSES IN SLABS AND BEAMS OF HOMOGENEOUS, ISOTROPIC, ELASTIC MATERIAL

Notation

S = free, unrestrained unit linear shrinkage-strain

$-S$ = free, unrestrained unit linear swelling-strain

S_∞ = final shrinkage-strain under fixed ambient conditions, value of S when $t = \infty$

S_{av} = average shrinkage over the volume of the specimen, the same as average shortening per unit length if the material follows Hooke's law

t = time in days

k = diffusivity coefficient of shrinkage in sq. in. per day

f = surface factor, characteristic of the material and the boundary conditions, in in. per day

a, b, c, d, l = distances related to the dimensions of the specimen in inches

$B = fb/k$, a non-dimensional parameter

$T = kt/b^2$, a non-dimensional parameter

B_c and T_c, non-dimensional parameters corresponding to B and T and used when a second characteristic dimension of the specimen must be considered

x, y, z = rectangular coordinates

β_n = nth root of $\beta \tan \beta = B$

β_m = same as β_n except used in connection with c, whereas β_n is used in connection with b

A_n = Fourier coefficient

$$F_n = \frac{2B}{B^2 + B + \beta_n^2}, \quad F_m = \frac{2B_c}{B_c^2 + B_c + \beta_m^2}$$

$$H_n = \frac{B}{\beta_n^2} F_n, \qquad H_m = \frac{B_c}{\beta_m^2} F_m$$

$$G_n = \left(\frac{1}{\cos \beta_n} - \frac{B}{2} - 1\right) \frac{F_n}{\beta_n^2}$$

σ_x, σ_y, σ_z = normal components of stress parallel to $x-$, $y-$, and $z-$axes—positive if tensile, negative if compressive.
e_x, e_y, e_z = elongations in $x-$, $y-$, and $z-$directions
τ_{xy}, τ_{xz}, τ_{yz} = shearing-stress components
γ_{xy}, γ_{xz}, γ_{yz} = shearing-strain components
E = Young's modulus in psi.
μ = Poisson's ratio
v = deflection in inches, displacement of the elastic line in the y-direction
N = the normal to the surface directed outward

$$P(x) = \frac{2}{\sqrt{\pi}} \int_0^x e^{-x^2} dx, \text{ the probability integral}$$

$$\phi(x) = \frac{2}{\sqrt{\pi}} \int_x^\infty e^{-x^2} dx = 1 - P(x)$$

$$\phi_b = 1 - \sum_1^\infty e^{-T\beta_n^2} F_n \frac{\cos \beta_n \frac{y}{b}}{\cos_e^6 \beta_n}$$

$$\phi_c = 1 - \sum_1^\infty e^{-T_c\beta_m^2} F_m \frac{\cos \beta_m \frac{z}{c}}{\cos \beta_m}$$

$$H_b = 1 - \sum_1^\infty e^{-T\beta_n^2} H_n$$

$$H_c = 1 - \sum_1^\infty e^{-T_c\beta_m^2} H_m$$

Equation for diffusion of unrestrained shrinkage

The diffusion equation is a mathematical statement of the fact that for each infinitesimal volume of a body the excess of the substance in question flowing in over that flowing out per unit of time is equal to the rate of increase of the substance in that volume. When similar assumptions are made in regard to shrinkage, shrinkage thus being treated as if it were a "substance" just as heat is so treated, the result is[2]

$$k\left[\frac{\partial^2 S}{\partial x^2} + \frac{\partial^2 S}{\partial y^2} + \frac{\partial^2 S}{\partial z^2}\right] = \frac{\partial S}{\partial t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where k is the diffusivity of shrinkage.

The equation becomes

$$k \frac{\partial S}{\partial N} = f(S_\infty - S) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

at exposed boundaries and

$$\frac{\partial S}{\partial N} = 0 \quad \text{(3)}$$

at sealed boundaries

where

N is the normal to the surface, directed away from the body

f is the surface factor

S_∞ is the value that S will eventually reach under fixed ambient conditions.

Equations 2 and 3 correspond, respectively, to Newton's law of cooling at exposed boundaries and to no flow of heat past perfectly insulated boundaries in the analogous problem of flow of heat.

If the boundaries of the body are not parallel planes, a transformation of Equation 1 from an expression in rectangular coordinates to some other form is usually desirable. For example, if the body is a circular cylinder, Equation 1 is best transformed to

$$k\left[\frac{\partial^2 S}{\partial r^2} + \frac{1}{r}\frac{\partial S}{\partial r} + \frac{1}{r^2}\frac{\partial^2 S}{\partial \Theta^2} + \frac{\partial^2 S}{\partial z^2}\right] = \frac{\partial S}{\partial t}$$

where r, Θ, and z are cylindrical coordinates. Frequently, the condition $\frac{\partial S}{\partial N} = 0$ at some boundaries or some other conditions will make S independent of certain coordinates and thereby simplify Equation 1.

Since the form of the solution for S depends upon the form of the differential equation, the form of the solution is dependent upon the boundary condition and the shape of the body under investigation. The initial conditions (values of S at $t = 0$) and any variation in boundary conditions with time will also affect the form of the solution.

Assumptions as to elastic properties

After a satisfactory solution for S has been obtained, then displacements and stresses will be found by the application of certain fundamentals of the theory of elasticity. The solutions for stresses are here restricted to homogeneous isotropic solids that follow Hooke's law. Also, as will be brought out below, the effect of Poisson's ratio will be neglected in some cases.

Effect of shape of body on relative values of principal stresses

The state of stress at any point in a body is defined by the directions and magnitudes of the three principal stresses. The three principal stresses in wide slabs and in narrow beams will be in the directions of length, width, and depth, respectively, if the bodies are under uniform exposure either from one or from two opposite faces and are without external restraint. The principal stress in the direction of depth (normal

to the exposed face) will be zero. The other two principal stresses will be equal in a wide slab (width large compared to depth) if the slab is either not restrained or is restrained equally in the directions of length and width; but in a narrow beam (width small compared to depth) the principal stress in the direction of width will be negligible, and the stress in the direction of length will be $(1 - \mu)$ times the corresponding stress in a corresponding slab. A corresponding slab differs from the beam just described primarily only in the matter of width. Beams whose widths are not small compared to their depths will have longitudinal stresses intermediate between those of slabs and narrow beams. Corresponding slabs and beams would, of course, have the same longitudinal stresses regardless of widths if Poisson's ratio were zero. Mathematical analyses will be made for the three cases shown in Fig. 1.*

To simplify the mathematical work the effects of Poisson's ratio will be neglected in most of the derivations for Cases I and II. When these effects are neglected, the results will be strictly correct only if Poisson's ratio is zero or if the beam is very narrow.

CASE I—SLAB OR BEAM DRYING FROM ONE FACE ONLY

Shrinkage

Solution by Fourier series. The exposed face of the slab or beam will be taken as the plane $y = b$, and the opposite face will be taken as the plane $y = 0$ as shown in Fig. 1 for Case I. For this case the diffusion equation reduces to

$$k \frac{\partial^2 S}{\partial y^2} = \frac{\partial S}{\partial t} \quad \text{(1a)}$$

The equation becomes

$$\frac{\partial S}{\partial y} = \frac{f}{k} (S_\infty - S) \quad \text{(2a)}$$

at the exposed boundary $y = b$ and

$$\frac{\partial S}{\partial y} = 0 \quad \text{(3a)}$$

at the sealed boundary $y = 0$.

A general solution for S satisfying Equations 1a and 3a is

$$S = S_\infty - \sum_1^\infty A_n \, e^{-\frac{kt}{b^2}\beta_n^2} \cos \beta_n \frac{y}{b} \quad \text{(4)}$$

*All figures and tables pertaining to Part I will be found on pages 196 to 204.

Equation 2a is also satisfied if β_n is the nth root of

$$\beta \tan \beta = \frac{fb}{k} \quad \text{.....(2b)}$$

i.e.,

$$\beta_n \tan \beta_n = \frac{fb}{k} \quad \text{.....(2c)}$$

The above statements may be verified by substituting S from Equation 4 into Equations 1a, 2a and 3a.

For time $t = \infty$, Equation 4 reduces to $S = S_\infty$, which is in accord with the definition of S_∞. An infinite series of terms such as the trigonometric series in Equation 4 is necessary to give an arbitrary distribution of shrinkage at time $t = 0$.

If the initial conditions are such that $S = 0$ when $t = 0$, then the Fourier coefficients A_n are given by*

$$A_n = \frac{S_\infty}{\cos \beta_n} \frac{2\frac{fb}{k}}{\left(\frac{fb}{k}\right)^2 + \frac{fb}{k} + \beta_n^2}$$

It therefore follows that

$$\frac{S}{S_\infty} = 1 - \sum_1^\infty e^{-T\beta_n^2} F_n \frac{\cos \beta_n \frac{y}{b}}{\cos \beta_n} \quad \text{.....(5)}$$

where

$$T = \frac{kt}{b^2}$$

$$F_n = \frac{2B}{B^2 + B + \beta_n^2}$$

$$B = \frac{fb}{k}$$

Equation 5 (in slightly different form) and similar equations for other shapes and other conditions, applied to analogous phenomena, may be found in the literature of mathematical physics such as the textbooks of Byerly, Carslaw, and Ingersoll and Zobel. Various tables and diagrams have been prepared from which the numerical relationship of the four non-dimensional quantities S/S_∞, y/b, B, and T may be found, such as Fig. 4, page 841 of Perry's *Chemical Engineer's Handbook* (1934).

*The general procedure of obtaining Fourier coefficients to satisfy initial conditions somewhat analogous to the present problem is given in Articles 66 to 68 of Byerly's *Fourier Series and Spherical Harmonics* Boston: Ginn & Co., 1893).

To use more than a few of the terms in Equation 5 for the evaluation of S/S_∞ is very laborious because of the difficulty in evaluating β_n and F_n. The number of terms required for a given degree of precision will depend somewhat upon the parameters B and y/b but is chiefly controlled by the parameter T. Computations show that very little error is introduced by neglecting all terms in the series except the first if T is more than about 0.2; but many terms are needed for the usually desired precision if T is less than 0.02,—the smaller the value of T the greater the number of terms needed. Very precise values of S/S_∞ for small values of T may be found without the tedious computation indicated in Equation 5 by using another expression which will now be derived.

Solution in terms of the probability integral. As long as the shrinkage at the sealed surface remains negligible, the distribution of shrinkage from the exposed surface inward will be nearly independent of the distance between the two surfaces. Suppose that instead of considering the surface at $y = 0$ to be sealed, the body is considered to be extended to infinity in a negative y-direction. Then instead of the boundary condition $\frac{\partial S}{\partial y} = 0$ at $y = 0$, the requirement will be

$$S = 0 \qquad (6)$$

at $y = -\infty$.

The solution* satisfying Equations 1a, 2a and 6 and giving $S = 0$ when $t = 0$ is

$$\frac{S}{S_\infty} = \phi\left[\frac{1 - \frac{y}{b}}{2\sqrt{T}}\right] - \phi\left[\frac{1 - \frac{y}{b}}{2\sqrt{T}}\right] e^{B\left(1 - \frac{y}{b}\right) + B^2T} \qquad (7)$$

where $\phi(x)$ is $\frac{2}{\sqrt{\pi}} \int_x^\infty e^{-x^2}\, dx$ and T is again used in place of $\frac{kt}{b^2}$.

The quantity $1 - \phi(x)$, or $\frac{2}{\sqrt{\pi}} \int_0^x e^{-x^2}\, dx$, is known as the probability integral. Values of $\phi(x)$ may be readily found by using a table of the probability integral.

Numerical calculations show that Equation 7 gives values that differ from those given by Equation 5 by an amount less than the value of S/S_∞ at $y = 0$; therefore, Equation 7 may be used in place of Equation 5 whenever T is so small that S/S_∞ at $y = 0$ is less than the permissible error.

*This solution is very similar to that given for an analogous problem by Carslaw in Article 25 of *The Conduction of Heat*, (Macmillan & Co., Ltd., 2d ed., 1921).

Table 2 and Fig. 7, showing S/S_∞ in terms of y/b, kt/b^2, and fb/k, were prepared from Equations 5 and 7.

Stresses and strains

Continuity, Hooke's law and equilibrium. As stated previously, the solutions for stresses are here restricted to homogeneous, isotropic solids that follow Hooke's law. Equations for the stresses that would be produced in such a body by the shrinkages given by Equations 5 or 7 will now be derived.

The shrinkage S has been defined as the linear unit deformation that would occur if each infinitesimal element were unrestrained. However, the properties of a continuous solid will not permit an arbitrary distribution of deformations; therefore, unless the distribution of shrinkage given by Equation 5 happens to be compatible with the conditions of continuity, stresses will be produced that will modify the deformations so as to make them compatible. Although in general six partial differential equations are required for a complete mathematical statement of the conditions of compatibility,[3] these are reduced to

$$\frac{\partial^2 e_x}{\partial y^2} = 0 \qquad (8)$$

for either long narrow beams (plane stress) or slabs (plane strain) if the stresses are considered to be independent of the longitudinal coordinate x.

The term e_x is defined as the resultant unit deformation in the x-direction (the direction of length). It is therefore the algebraic sum of shrinkage, S, and the strain produced by stresses. σ_y is obviously zero; and if Poisson's ratio is zero or if the discussion is confined to narrow beams, σ_z is negligible. Therefore,

$$e_x = \frac{\sigma_x}{E} - S \qquad (9)$$

or, solving for stress,

$$\sigma_x = E\,(e_x + S) \qquad (10)$$

where E is Young's modulus.

The restriction imposed by Equation 8 requiring that the expression for longitudinal deformation contain no terms in y other than the first power (second derivative equal to zero) is equivalent to the assumption usually made in the elementary theory of beams that "plane cross-sections remain plane." If longitudinal restraint is complete, then e_x is zero and it follows from Equation 10 that $\sigma_x = ES$. If, however, longitudinal shortening is permitted but complete restraint against bending is provided, then e_x is not zero but is still independent of y. If the beam has no external restraint, the non-symmetrical distribution of shrinkage causes it to warp, making e_x a linear function of y. For no external restraint the equations of

equilibrium (summation of forces in the x-direction equal to zero and summation of moments about the z-axis equal to zero) become

$$\int_0^b \sigma_x \, dy = 0 \quad \ldots\ldots (11)$$

and

$$\int_0^b \sigma_x y \, dy = 0 \quad \ldots\ldots (12)$$

It may be shown by substituting Equation 10 into both Equations 11 and 12 that if shrinkage (S) is either independent of or a linear function of y, an unrestrained beam will be free of stress ($e_x = -S$ and $\sigma_x = 0$). For any other variation of shrinkage a stressed condition must result because the restriction on e_x (Equation 8) will not permit it to be equal and opposite to S if shrinkage is a non-linear function of y.

The only solution for e_x that satisfies Equations 8, 10, 11 and 12 is

$$e_x = \left(6\frac{y}{b} - 4\right)\frac{1}{b}\int_0^b S \, dy + \left(6 - 12\frac{y}{b}\right)\frac{1}{b^2}\int_0^b Sy \, dy. \quad (13)$$

When this value of e_x is substituted into Equation 10, the result is

$$\sigma_x = E\left[S + \left(6\frac{y}{b} - 4\right)\frac{1}{b}\int_0^b S \, dy + \left(6 - 12\frac{y}{b}\right)\frac{1}{b^2}\int_0^b Sy \, dy\right] \quad (14)$$

Finally, S from Equation 5 may be substituted into Equation 14, thus giving stress in an unrestrained beam as a function of the parameters y/b, kt/b^2, fb/k, S_∞ and of Young's modulus. This substitution will not be made until later, because it seems advisable at this time to consider another approach.

Solution by superposition. Although the above derivation is short and is in the simplest form for checking the mathematical correctness of the equation, a derivation in which elementary solutions are superposed is also desirable because it will be easier in general to understand and because the final expressions are in more usable forms. In this second derivation the resultant stress σ_x is considered as consisting of three parts. The first part is that stress which would be produced by complete restraint against longitudinal deformation; the second part is a uniform stress equal to and opposite in sign to the average of the first part; the third part is a stress

resulting from a simple moment that is equal to and opposite in sign to the moment produced by the sum of the first two parts. That is, the first part alone σ_x' would result from complete restraint, the sum of the first and second parts σ_x'' would result from restraint against warping only; the sum of all three parts, i.e., σ_x, would result if no external restraint were applied during shrinkage.

Although in this derivation an expression for σ_x appears to be the ultimate goal, expressions for σ_x' and for σ_x'' are also desirable. The stress σ_x' may be representative of the stress in pavement slabs or building walls that are restrained from shortening and the stress σ_x'' is representative of an unrestrained wall drying equally from two opposite sides (Case II discussed later).

Since for complete longitudinal restraint $e_x = 0$, it follows from Equation 10 that the first part of the stress is

$$\sigma_x' = ES \qquad (15)$$

Since the average value of σ_x' is $\frac{1}{b}\int_0^b \sigma_x'\, dy$, the second part of σ_x is $-\frac{E}{b}\int_0^b S\, dy$; therefore, the sum of the first and second parts (σ_x'') is given by

$$\sigma_x'' = E\left[S - \frac{1}{b}\int_0^b S\, dy\right] \qquad (16)$$

The moment produced by σ_x'' is the moment necessary to prevent warping. This moment per unit width of beam is found by multiplying Equation 16 by $y\, dy$ and integrating. This gives

$$M = \int_0^b \sigma_x''\, y\, dy = E\left[\int_0^b Sy\, dy - \frac{b}{2}\int_0^b S\, dy\right] \qquad (17)$$

For no external restraint this moment must be removed by superposing an equal and opposite moment. The stress resulting from a moment $-M$ is given by the elementary theory of beams as

$$\frac{M\,(y - b/2)}{1/12\, b^3}$$

This becomes

$$E\left(6 - 12\frac{y}{b}\right)\left[\frac{1}{b^2}\int_0^b Sy\,dy - \frac{1}{2b}\int_0^b S\,dy\right]$$

when M from Equation 17 is substituted. When this stress, the third part of σ_x is added to the sum of the first and second parts of Equation 16, the result is Equation 14 previously derived.

Stresses in terms of shortening and warping. The shortening of the beam can be considered as due to the second part of the stress since it is the addition of this part that removes longitudinal restraint and thereby permits the shortening of the beam. From these considerations it follows that

$$\text{unit shortening} = S_{av} = \frac{1}{b}\int_0^b S\,dy \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

where S_{av} is the average value of S.

The bending (warping) of the beam can be considered as due to the third part of the stress since it is the addition of this part that removes the remaining external restraint and thereby permits warping of the beam. The deflection v caused by the moment $-M$ is given by the elementary theory of the bending of beams as $v = \dfrac{6Mx\ (l - x)}{Eb^3}$ where l is the span and v is the deflection of points within the span with respect to either of the end points $x = 0$ and $x = l$. The maximum deflection v_{max} (warping) that occurs at $x = l/2$ is therefore

$$v_{max} = \frac{6M}{Eb^3}\left(\frac{l}{2}\right)^2 = \frac{3l^2}{2b}\left[\frac{1}{b^2}\int_0^b Sy\,dy - \frac{1}{2b}\int_0^b S\,dy\right]^* \quad \ldots\ldots (19)$$

By writing the second part of σ_x, $-\dfrac{E}{b}\displaystyle\int_0^b S\,dy$, in terms of the unit shortening S_{av} it produces, and by writing the third part of σ_x,

$$E\left(6 - 12\frac{y}{b}\right)\left[\frac{1}{b^2}\int_0^b S\,y\,dy - \frac{1}{2b}\int_0^b S\,dy\right],$$

*The part in the brackets is numerically equal to one-sixth of the unit strain ϵ that would be produced in either the top or the bottom of the beam by a moment just sufficient to straighten it. That is,

$$\frac{1}{6}\epsilon = \frac{2bv_{max}}{3l^2}$$

in terms of the deflection v_{max} it produces, the expressions for the stresses are put into more usable forms. When this is done, the following equations for stresses are obtained.

For complete longitudinal restraint (first part of σ_x),

$$\sigma_x' = ES \quad \ldots\ldots (15)$$

For restraint against warping only (sum of first and second parts of σ_x),

$$\sigma_x'' = E\,(S - S_{av}) \quad \ldots\ldots (20)$$

For no external restraint (sum of all three parts of σ_x),

$$\sigma_x = E\left[S - S_{av} + \left(6 - 12\frac{y}{b}\right)\frac{2bv_{max}}{3l^2}\right] \quad \ldots\ldots (21)$$

Evaluation of the parameters $\dfrac{S_{av}}{S_\infty}$ *and* $\dfrac{2bv_{max}}{3l^2S_\infty}$. When Equation 5 for S/S_∞ is substituted in Equation 18 for shortening and in Equation 19 for warping and the indicated integrations are performed, the result is

$$\frac{S_{av}}{S_\infty} = \frac{1}{b}\int_0^b \frac{S}{S_\infty}\,dy = 1 - \sum_1^\infty e^{-T\beta_n^2} H_n \quad \ldots\ldots (22)$$

$$\frac{2bv_{max}}{3l^2S_\infty} = \frac{1}{b^2}\int_0^b \frac{S}{S_\infty}\,y\,dy - \frac{S_{av}}{2S_\infty} = \sum_1^\infty e^{-T\beta_n^2} G_n \quad \ldots\ldots (23)$$

where

$$H_n = \frac{2B^2}{\beta_n^2\,(B^2 + B + \beta_n^2)}$$

and

$$G_n = \left(\frac{1}{\cos\beta_n} - \frac{B}{2} - 1\right)\frac{F_n}{\beta_n^2}$$

If T, the non-dimensional time-factor, is small, the series in Equations 22 and 23 converge rather slowly, and in that case it is convenient to use the following equations obtained by substituting Equation 7 into Equations 18 and 19, respectively.*

$$\frac{S_{av}}{S_\infty} = \frac{1}{B}\left[e^{B^2T}\,\phi(B\sqrt{T}) - 1 + \frac{2B\sqrt{T}}{\sqrt{\pi}}\right] \quad \ldots\ldots (24)$$

*The lower integration limit for each integral is decreased from 0 to $-\infty$.

$$\frac{2bv_{max}}{3l^2S_\infty} = \left\{\left[\frac{1}{2B} + \frac{1}{B^2}\right]\left[e^{B^2T}\ \phi(B\sqrt{T}) - 1 + \frac{2B\sqrt{T}}{\sqrt{\pi}}\right] - T\right\} \quad \ldots (25)$$

Furthermore, if the parameter $B\sqrt{T}$ is very small, it is still better to use the following equations obtained by expanding the expressions in the brackets of Equations 24 and 25.

$$\frac{S_{av}}{S_\infty} = BT\left[1 - \frac{4}{3\sqrt{\pi}} B\sqrt{T} + \frac{1}{2}(B\sqrt{T})^2 - \frac{8}{15\sqrt{\pi}}(B\sqrt{T})^3 + \ldots\right] \quad (24a)$$

$$\frac{2bv_{max}}{3l^2S_\infty} = \left[\frac{BT}{2} - T\left(1 + \frac{B}{2}\right)\left(\frac{4}{3\sqrt{\pi}} B\sqrt{T} - \frac{1}{2}(B\sqrt{T})^2 + \frac{8}{15\sqrt{\pi}}(B\sqrt{T})^3\right) - \ldots\right] \quad \ldots (25a)$$

In general the following rules will be found applicable for rapid evaluation of the parameters $\frac{S_{av}}{S_\infty}$ and $\frac{2bv_{max}}{3l^2S_\infty}$ to a fair degree of accuracy.

If T is more than about 0.05, use Equations 22 and 23.

If T is less than about 0.05 and B is more than about 5, use Equations 24 and 25.

If T is less than about 0.05 and B is less than about 5, use Equations 24a and 25a.

Forces and moments necessary for complete restraint. The force necessary for longitudinal restraint is $\int \sigma_x'\, dA$. Therefore, the average force per unit area is $\frac{1}{b}\int_0^b \sigma_x'\, dy$. From Equations 15, 5, and 22 this becomes

$$\text{force per unit area} = ES_\infty\left[1 - \sum_1^\infty e^{-T\beta_n^2} H_n\right] \quad \ldots (26)$$

From Equations 17, 5, and 23, the moment per unit width necessary for restraint against warping is found to be

$$M = ES_\infty b^2 \sum_1^\infty e^{-T\beta_n^2} G_n \quad (27)$$

Simplification by taking B as equal to infinity. The principal equations derived above reduce to simpler forms and the computation of numerical values is less tedious if the assumption is made that B, i.e., fb/k, equals infinity. If B is large, say 100 or more, the error introduced by assuming it to be infinity is negligible. However, if B is less than about 5, the error introduced by considering it to be infinity may be appreciable as is shown, for example, by Fig. 8, 9, and 14. Whether justifiable or not, the assumption that $B = \infty$ is frequently made in analogous problems to which the diffusion equation applies. This assumption was made by Terzaghi and Fröhlich[4] in developing the theory of settlement of foundations due to consolidation of underlying material, by Glover[5] in a study of distribution of temperature in concrete dams, and by Carlson[1] in a study of distribution of moisture and shrinkage in concrete. The more important of the above equations for the special case of $B = \infty$ are given below:

Equation 5 becomes

$$\frac{S}{S_\infty} = 1 - \sum_1^\infty \frac{4\,(-1)^{n-1}}{(2n-1)\pi}\, e^{-(2n-1)^2 \frac{\pi^2}{4} T} \cos\,(2n-1)\frac{\pi y}{2b}.$$

Equation 7 becomes

$$\frac{S}{S_\infty} = \phi\left(\frac{1 - y/b}{2\sqrt{T}}\right).$$

Equation 22 becomes

$$\frac{S_{av}}{S_\infty} = 1 - \frac{8}{\pi^2} \sum_1^\infty \frac{1}{(2n-1)^2}\, e^{-(2n-1)^2\,(\pi^2/4)\,T}$$

Equation 24 becomes

$$\frac{S_{av}}{S_\infty} = \frac{2}{\sqrt{\pi}}\sqrt{T}.$$

Equation 23 becomes

$$\frac{2bv_{max}}{3l^2 S_\infty} = \sum_1^\infty \frac{4}{\pi^2\,(2n-1)^2}\left[\frac{(-1)^{n-1}\,4}{\pi\,(2n-1)} - 1\right] e^{-(2n-1)^2 \frac{\pi^2}{4} T}.$$

Equation 25 becomes

$$\frac{2bv_{max}}{3l^2 S_\infty} = \frac{\sqrt{T}}{\sqrt{\pi}} - T.$$

*Tables, curves, and computations.** Tables and diagrams such as those by Newman[6] are available from which values of S/S_∞ and S_{av}/S_∞ may be determined. However, such published tables are in general not adequate for the present problem. The smallest value of the parameter T used by Newman in his computations was 0.1, whereas the stresses in concrete may be desired for a much earlier period. The tables given here were prepared for T as low as 0.001. Moreover, so far as is known, the parameter $\frac{2bv_{max}}{3l^2S_\infty}$ had not previously been evaluated for this or any analogous problem and its evaluation is necessary for the problem here considered.

A step in the evaluation of Equations 15, 20, and 21 for the theoretical stresses in a beam drying from one side under the different modes of restraint is the evaluation of the three quantities $\frac{S}{S_\infty}$, $\frac{S_{av}}{S_\infty}$ and $\frac{2bv_{max}}{3l^2S_\infty}$. The first quantity S/S_∞ as a function of the three parameters y/b, B and T, is given in Table 2 and shown graphically in Fig. 7. The second quantity S_{av}/S_∞ as a function of B and T is given in Table 3 and shown graphically in Fig. 8.† The third quantity $\frac{2bv_{max}}{3l^2S_\infty}$ as a function of B and T is given in Table 4 and shown graphically in Fig. 9. Tables 5 and 6 giving the stresses σ_x'' and σ_x (Equations 20 and 21) as functions of the three parameters y/b, B, and T, were readily prepared after the three primary quantities had been evaluated (Tables 2, 3, and 4). Results for $B = 5$ are shown graphically in Fig. 10, 11, 12, and 13. Fig. 14 shows maximum values of $\frac{\sigma_x''}{ES_\infty}$ and $\frac{\sigma_x}{ES_\infty}$ and of $\frac{2bv_{max}}{3l^2S_\infty}$ versus the parameter B.

The computations made for the preparation of the tables and diagrams are explained in part below.

If the parameter T is so small that the equations based upon the assumption that the body extends to infinity may be used instead of the theoretically correct equations, no difficulty is encountered. For example, let $y/b = 0.8$, $B = 5$, and $T = 0.01$. Equation 7 then becomes

$$\frac{S}{S_\infty} = \phi\left(\frac{0.2}{2 \times 0.1}\right) - \phi\left(\frac{0.2}{2 \times 0.1} + 5 \times 0.1\right) e^{5 \times 0.2 + 25 \times 0.01}$$

$$= \phi\,(1) - \phi\,(1.5)\, e^{1.25}$$

*The values of β_n given in Table 1 differ slightly from those given by Newman in some instances. The values in Table 1 are believed to be accurate to the number of places given. The tables other than Table 1 have not been checked by duplicate computations. But, except for the last digit, which may be inaccurate by a point or two, these tables are believed to be reasonably accurate.

†In Fig. 8 and 9, where shortening and warping were the dependent variables, the square root of T for the abscissas was found to be better than T, because a considerable part of such graphs were approximately straight lines. For this reason the square root of T rather than T was used for the abscissas in the construction of other diagrams.

From mathematical tables

$$\phi\,(1) = 0.15730; \phi\,(1.5) = 0.03389; e^{1.25} = 3.4903$$

Therefore $S/S_\infty = 0.1573 - 0.03389 \times 3.4903 = 0.0390$.

Note that this is the value given in Table 2 for the above values of B, T, and y/b. Also note that for the same B and T the table gives zero for $y/b = 0$, showing that it was permissible to use Equation 7 instead of Equation 5.

When the theoretically correct equations are used, the computations are more involved. For instance, let $T = 0.1$ instead of 0.01 in the above example. T will then be so large that S/S_∞ will have an appreciable value at $y/b = 0$. Therefore, Equation 7 will not be applicable and Equation 5, the exact equation, must be used. A substitution of values for T and y/b into Equation 5 gives

$$\frac{S}{S_\infty} = 1 - \sum_{1}^{\infty} e^{-0.1\beta_n^2} F_n \frac{\cos 0.8\beta_n}{\cos \beta_n}$$

The first step in evaluating the above expression is to determine β_n which Equation 2c shows to be a function of fb/k and n, i.e., B, and the integer n. The determination of β_n by interpolation is simplified by the introduction of α_n where α_n depends on B and n. The equation for β_n is then written

$$\beta_n = (n - 1 + \alpha_n)\pi \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Curves of α_n versus B for the first six values of n and for $n = 21$ are shown in Fig. 2. By means of Fig. 2 and Equation 28 any desired β_n may be found with reasonable accuracy for any value of B. The first six values of β_n for several different values of B are given in Table 1.

After finding β_n for the given values of B and n, the factors F_n, $\cos \beta_n$, $\cos\left(\beta_n \frac{y}{b}\right)$ and $e^{-T\beta_n^2}$ are determined. F_n and $\cos \beta_n$ as functions of B are shown in Fig. 3 and 4, respectively, for the first six values of n. The functions $\cos\left(\beta_n \frac{y}{b}\right)$ and $e^{-T\beta_n^2}$ are readily obtained from mathematical tables after the products $\beta_n \frac{y}{b}$ and $T\beta_n^2$ have been determined. When the proper values of the four factors listed above are substituted, the above equation for $\frac{S}{S_\infty}$ becomes

$$\frac{S}{S_\infty} = 1 - \frac{0.84147 \times 0.3152 \times 0.4966}{0.2541} - \frac{0.1965 \times 0.2161 \times 0.9963}{0.6277}$$
$$- \frac{0.00844 \times 0.1286 \times 0.7280}{0.8101} + \ldots$$

This reduces to $\frac{S}{S_\infty} = 1 - 0.5183 - 0.0674 - 0.0009 = 0.4134$

The values for S/S_∞ given in Table 2 were computed by one or the other of the methods illustrated above.

In preparing Table 3 from which Fig. 8 was constructed (Fig. 8 shows shortening as a function of $\sqrt{T}$ for various values of B), values of H_n in Equation 22 were needed. Values of H_n as a function of B for the first six values of n are shown in Fig. 5. In like manner, Fig. 6 showing G_n as a function of B, served in the preparation of Table 4 from which Fig. 9 was constructed. Of course, for small values of T, Fig. 5 and 6 are not necessary since either Equation 24 or Equation 24a is used instead of Equation 22 and either Equation 25 or Equation 25a is used instead of Equation 23, depending on the value of B.

Application to beams or slabs of any width-to-depth ratio when Poisson's ratio is not zero. The effect of Poisson's ratio was neglected in the preceding derivations. Its effect stated in general terms in the introductory remarks in regard to Case I will now be analyzed in more detail. If Poisson's ratio is not zero, Equation 9 for e_x and Equation 10 for σ_x will be modified to include the effect of σ_z. That is,

$$e_x = \frac{\sigma_x}{E} - \frac{\mu\sigma_z}{E} - S \quad \ldots\ldots (9a)$$

$$\sigma_x - \mu\sigma_z = E\,(e_x + S) \quad \ldots\ldots (10a)$$

However, if the ratio of width to depth is small, σ_z will be negligible and Equation 10a reduces to Equation 10. On the other hand, if the ratio of width to depth is very large (a slab), the width being comparable with the length, then σ_z will be equal to σ_x. If $\sigma_x = \sigma_z$, then Equation 10a reduces to

$$\sigma_x = \frac{E}{1-\mu} \cdot (e_x + S) \quad \ldots\ldots (10b)$$

The only difference between Equation 10b for a wide slab and Equation 10 for a narrow beam is the factor μ, which occurs in Equation 10b but not in Equation 10. Therefore, for stresses in a slab, E in Equations 15, 20, and 21 is replaced by $\frac{E}{1-\mu}$. The stresses in beams whose width-to-depth ratio is intermediate will have stresses intermediate between those of narrow beams and of slabs. Since E does not appear in Equa-

tion 22 for average shrinkage nor in Equation 23 for warping, these quantities are the same for narrow beams and wide slabs.

CASE II—SLAB OR BEAM DRYING FROM TWO OPPOSITE SURFACES

Equations taken from those derived for Case I. Since the flow of moisture in a slab drying from only one surface is believed to be the same as that in either half of a slab of twice the depth drying from two opposite surfaces, it will be assumed that the theoretical equations derived for shrinkage of a beam or slab drying from only one surface will apply equally well for either half of a beam or slab drying from two opposite surfaces. The plane midway between the drying surfaces will be taken as the plane $y = 0$ as shown in Fig. 1 for Case II. Since the two halves of the beam will mutually restrain each other from warping, the equations for stresses, strains, and shortening in each half will be the same as those given previously in Case I for a beam restrained against warping and drying from one surface.

CASE III—RECTANGULAR PRISM DRYING FROM FOUR FACES

Shrinkage

The differential equation and boundary conditions. For a prism drying from four faces but not from the ends the diffusion equation reduces to

$$k\left(\frac{\partial^2 S}{\partial y^2} + \frac{\partial^2 S}{\partial z^2}\right) = \frac{\partial S}{\partial t} \qquad (1b)$$

The exposed faces of the prism will be taken as the planes $y = \pm b$, $z = \pm c$, as shown in Fig. 1 for Case III. The boundary conditions then become

$$\frac{\partial S}{\partial y} = \pm \frac{f}{k}(S_\infty - S) \qquad (2a)$$

at the boundaries $y = \pm b$ and

$$\frac{\partial S}{\partial z} = \pm \frac{f}{k}(S_\infty - S) \qquad (3b)$$

at the boundaries $z = \pm c$.

The solution satisfying Equations 1b, 2a, 3b, and giving $S = 0$ at $t = 0$ and $S = S_\infty$ at $t = \infty$ is the following:

$$\frac{S}{S_\infty} = 1 - \left[\sum_1^\infty e^{-T\beta_n^2} \frac{\cos \beta_n \frac{y}{b}}{F_n \cos \beta_n}\right]\left[\sum_1^\infty e^{-T_c \beta_m^2} \frac{\cos \beta_m \frac{z}{c}}{F_m \cos \beta_m}\right] \qquad (5a)$$

where β_m, F_m, and T_c correspond to β_n, F_n, and T, respectively, the difference being that the dimension b has been replaced by c.

Shrinkage expressed in terms of the solutions given for a prism drying from one face or two opposite faces. Since the infinite series in the first bracket in Equation 5a is identical with the one given in Equation 5 and the infinite series in the second bracket is like the first except that y is replaced by z, b by c, etc., and since Equation 5 applies to either half of a slab exposed on two opposite surfaces (Case II), it follows that the brackets have the following values:

$$\Sigma e^{-T\beta_n^2} F_n \frac{\cos \beta_n \frac{y}{b}}{\cos \beta_n} = 1 - \phi_b \quad \ldots\ldots (29)$$

$$\Sigma e^{-T_c\beta_m^2} F_m \frac{\cos \beta_m \frac{z}{c}}{\cos \beta_m} = 1 - \phi_c \quad \ldots\ldots (30)$$

where ϕ_b is the value S/S_∞ would have if only the surfaces $y = \pm b$ were exposed and ϕ_c is the value S/S_∞ would have if only the surfaces $z = \pm c$ were exposed.

A substitution of Equations 29 and 30 into Equation 5a gives

$$\frac{S}{S_\infty} = \phi_b + \phi_c - \phi_b\phi_c \quad \ldots\ldots (5b)$$

Equation 5b shows that the evaluation of shrinkage for a prism drying from four surfaces becomes a problem of adding the independent effects of drying from surfaces that are perpendicular to each other and then subtracting a term proportional to the product of the separate effects.

For example, consider the shrinkage tendency at the point $y = 0.4b$, $z = 0.8c$ in a prism for which $c = 2b$ (width equal to twice the thickness). Let f, k, and t be such that $fb/k = 5.0$ and $kt/b^2 = 0.20$; then $fc/k = 10.0$ and $kt/c^2 = 0.05$. ϕ_b is found in Table 2 or from Fig. 7 to be 0.2398. Since Table 2 was prepared for B equal to 0.1, 1.0, 5.0, and ∞ only, and since $fc/k = 10$, ϕ_c can be found from Table 2 only by interpolation. However, examination of Table 2 indicates that for kt/c^2 equal to 0.05 Equation 7 can be used instead of Equation 5 without appreciable error and therefore the equation rather than the table will be used to obtain ϕ_c. From Equation 7

$$\phi_c = \phi\left(\frac{0.2}{2\sqrt{0.05}}\right) - \phi\left(\frac{0.2}{2\sqrt{0.05}} + 10\sqrt{0.05}\right) e^{2+5}.$$

From tables giving probability integrals and the exponential function

$$\phi_c = 0.5273 - 0.000156 \times 1097 = 0.355.$$

Therefore at $y = 0.4b$, $z = 0.8c$, and $t = 0.20\ b^2/k$,

$$\frac{S}{S_\infty} = 0.2398 + 0.355 - 0.2398 \times 0.355 = 0.510.$$

Shortening expressed in terms of the shortening of a prism drying from one face or from two opposite faces. The average shrinkage S_{av} is given by

$$S_{av} = \frac{1}{bc} \int_0^b \int_0^c S \, dy \, dz \quad \text{.....(18a)}$$

From Equations 5a and 18a

$$\frac{S_{av}}{S_\infty} = 1 - \sum_1^\infty e^{-T\beta_n^2} H_n \sum_1^\infty e^{-T_c \beta_m^2} H_m \quad \text{.....(22a)}$$

or

$$\frac{S_{av}}{S_\infty} = H_b + H_c - H_b H_c, \quad \text{.....(22b)}$$

where H_b is the value S_{av}/S_∞ would have if only the surface $y = \pm b$ were exposed and H_c is the value S_{av}/S_∞ would have if only the surfaces $z = \pm c$ were exposed*. Therefore, the average shrinkage, and consequently the shortening, if the body is elastic, of a prism drying from four sides may be found by considering the separate effects of drying from opposite sides in pairs.

For example, consider the shortening of the prism discussed above. H_b is found in Table 3 to be 0.3510 and H_c is found to be 0.1753. Therefore,

$$\frac{S_{av}}{S_\infty} = 0.3510 + 0.1753 - 0.3510 \times 0.1753 = 0.4647.$$

Stresses and strains

Nature of the problem and the method to be used to obtain a solution. In Cases I and II previously discussed, where shrinkage was a function of time t and only one space coordinate y and where the problem was further simplified by neglecting the effects of the length and the width of the specimen on the distribution of stresses, a solution for the one stress involved was readily obtained. However, in the problem now under consideration, a prism drying from four surfaces, shrinkage varies with an additional coordinate z. As a result stresses vary with this additional coordinate also, and more than one stress will be involved. The problem will be somewhat simplified by neglecting the effect of the length on the distribution of stresses, i.e., the assumption will be made that stresses do not vary along the length. The distribution of stresses given by the solution based on this assumption will deviate a negligible amount from

*If a prism were drying from all six surfaces, the corresponding equation would be $S_{av}/S_\infty = H_a + H_b + H_c - H_a H_b - H_b H_c - H_c H_a + H_a H_b H_c$. Another way of expressing these relations is given by Glover (Ref. 7).

the theoretically correct distribution when stresses in the central portion of a long prism are under consideration (principle of Saint Venant[8]).

The solution for shrinkage in terms of two space coordinates was almost as simple as when only one coordinate was involved (Equation 5a for Case III compared with Equation 5 for Cases I and II), because tendency to shrink is considered to be a scalar quantity. On the other hand, since stresses are tensor quantities, the solution for stresses usually becomes much more complicated whenever more than one coordinate is involved. In fact, elasticians have obtained exact solutions meeting all boundary conditions for only a relatively few problems in which stresses were functions of at least two coordinates and then only by considering the body to be infinite in the direction of one of these coordinates. The difficulty is that since stresses are tensors, boundary forces are vectors, and in two-dimensional problems two components of force must be satisfied at each boundary. The specified conditions of stress at any two opposite boundaries can be satisfied by superposing particular solutions of the differential equations in accordance with the usual methods of Fourier analysis. But, in general, solutions satisfying rigorously the boundary requirements at two pairs of opposite boundaries simultaneously cannot be found by the usual methods.

A method of solving such problems after the appropriate differential equations have been derived was explained by the author in a recent paper[9]. That method will be used here. It is about the same as that used previously by Taylor[10] and by Timoshenko[11] in analogous problems.

Derivation of the differential equations relating stresses to shrinkage. By neglecting the variation of stresses and strains along the length of a body the problem becomes a two-dimensional problem in plane strain. The following equations taken from the theory of elasticity are then applicable[12].

Equations of Equilibrium:

$$\frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{yz}}{\partial z} = 0$$

$$\frac{\partial \sigma_z}{\partial z} + \frac{\partial \tau_{yz}}{\partial y} = 0$$

Condition of Compatibility:

$$\frac{\partial^2 e_y}{\partial z^2} + \frac{\partial^2 e_z}{\partial y^2} = \frac{\partial^2 \tau_{yz}}{\partial y \partial z}$$

Hooke's Law Modified to Include Isotropic Shrinkage:

$$e_x = \frac{1}{E}\left[\sigma_y - \mu\sigma_y - \mu\sigma_z\right] - S = -S_{av} \text{ in this problem}$$

$$e_y = \frac{1}{E}[-\mu\sigma_x + \sigma_y - \mu\sigma_z] - S$$

$$e_z = \frac{1}{E}[-\mu\sigma_x - \mu\sigma_y + \sigma_z] - S$$

$$\gamma_{yz} = \frac{2\,(1+\mu)}{E}\,\tau_{yz}$$

The above seven equations giving relations between the eight unknown stresses and strains can be reduced to the following two equations by eliminating the four strains and the shear stress:

$$\sigma_x = \mu\,(\sigma_y + \sigma_z) + E\,(S - S_{av}) \qquad (31)$$

$$\nabla^2(\sigma_y + \sigma_z) = \frac{E}{1-\mu}\,\nabla^2 S \qquad (32)$$

where ∇^2 is written for $\dfrac{\partial^2}{\partial y^2} + \dfrac{\partial^2}{\partial y^2}$.

These two equations together with the two equations of equilibrium and the boundary conditions that

$$\sigma_y \text{ and } \tau_{yz} = 0 \text{ at } y = \pm b$$

$$\sigma_z \text{ and } \tau_{yz} = 0 \text{ at } z = \pm c$$

and Equation 5a for S constitute the mathematical statement of the problem.

Solution for stresses. In general the stress σ_x will be larger than either σ_y or σ_z. The stress τ_{yz} will be relatively small in all cases. If only the value of the theoretical maximum stress is desired, a fairly good approximation can be obtained by the following simplified formula:

$$\sigma_x = E\,(S - S_{av}) \text{ approx.} \qquad (31a)$$

where S is given by Equation 5a and S_{av} is given by Equation 22a. If, however, an accurate theoretical value of all stresses is desired, a complete solution must be obtained and this is given below:

The solution for σ_y, σ_z, and τ_{yz} meeting all of the above requirements is as follows:*

$$\sigma_y = A_o\,\frac{c^2 - 3z^2}{12bc} + \frac{b}{c}\sum_{i=1}^{\infty}\sum_{j=1}^{\infty}\frac{A_{ij}}{i^2}\cos\,\alpha_i y\,\cos\,\gamma_j z$$

*Equations 33, 34, and 35 for stresses and Equations 36 and 37 for the coefficients B_j and C_i given here are almost the same as Equations 1, 2, and 3 for stresses and Equations 6 and 7 for the coefficients B_n and A_n respectively given in Ref. 9. The A_{ij}-series and the A_o-terms enter into the equations given here in place of the terms in the stress S given there; otherwise, except for slight differences in notation, the corresponding equations are identical and the equations given here may be derived by the procedures given there.

Equation 38 for A_{ij} and Equation 39 for A_o may be verified by substituting Equations 33, 34, and 5a into Equation 32 and then proving the equality of the two sides of the resulting equation within the domain under consideration by the usual Fourier analysis.

$$+ \sum_{j=1}^{\infty} B_j \frac{\cos \gamma_j z}{\cosh \gamma_j b} [\gamma_j y \sinh \gamma_j y - (1 + \gamma_j b \coth \gamma_j b) \cosh \gamma_j y]$$

$$- \sum_{i=1}^{\infty} C_i \frac{\cos \alpha_i y}{\cosh \alpha_i c} [\alpha_i z \sinh \alpha_i z + (1 - \alpha_i c \coth \alpha_i c) \cosh \alpha_i z] \quad (33)$$

$$\sigma_z = A_o \frac{b^2 - 3y^2}{12bc} + \frac{c}{b} \sum_{i=1}^{\infty} \sum_{j=1}^{\infty} \frac{A_{ij}}{j^2} \cos \alpha_i y \cos \gamma_j z$$

$$- \sum_{j=1}^{\infty} B_j \frac{\cos \gamma_j z}{\cosh \gamma_j b} [\gamma_j y \sinh \gamma_j y + (1 - \gamma_j b \coth \gamma_j b) \cosh \gamma_j y]$$

$$+ \sum_{i=1}^{\infty} C_i \frac{\cos \alpha_i y}{\cosh \alpha_i c} [\alpha_i z \sinh \alpha_i z - (1 + \alpha_i c \coth \alpha_i c) \cosh \alpha_i z] \quad (34)$$

$$\tau_{yz} = \sum_{i=1}^{\infty} \sum_{j=1}^{\infty} \frac{A_{ij}}{ij} \sin \alpha_i y \sin \gamma_j z$$

$$+ \sum_{j=1}^{\infty} B_j \frac{\sin \gamma_j z}{\cosh \gamma_j b} [\gamma_j b \coth \gamma_j b \sinh \gamma_j y - \gamma_j y \cosh \gamma_j y]$$

$$+ \sum_{i=1}^{\infty} C_i \frac{\sin \alpha_i y}{\cosh \alpha_i c} [\alpha_i c \coth \alpha_i c \sinh \alpha_i z - \alpha_i z \cosh \alpha_i z] \quad \ldots\ldots (35)$$

where $\alpha_i = \dfrac{i\pi}{b}$, $\gamma_j = \dfrac{j\pi}{c}$

$$B_j = \frac{- \dfrac{c}{b} \dfrac{A_o (-1)^j}{\pi^2 j^2} + \dfrac{b}{c} \displaystyle\sum_{i=1}^{\infty} (-1)^i \frac{A_{ij}}{i^2} - \sum_{i=1}^{\infty} (-1)^{i+j} \frac{4}{j\pi} \frac{ic}{jb} \dfrac{\tanh \dfrac{i\pi c}{b}}{\left[1 + \left(\dfrac{ic}{jb}\right)^2\right]^2} C_i}{1 + \dfrac{j\pi b}{c} \left(\coth \dfrac{j\pi b}{c} - \tanh \dfrac{j\pi b}{c} \right)} \quad \ldots\ldots (36)$$

$$C_i = \frac{-\frac{c}{b}\frac{A_o(-1)^i}{\pi^2 i^2} + \frac{c}{b}\sum_{j=1}^{\infty}(-1)^j\frac{A_{ij}}{j^2} - \sum_{j=1}^{\infty}(-1)^{i+j}\frac{4}{i\pi}\frac{jb}{ic}\frac{\tanh\frac{j\pi b}{c}}{\left[1+\left(\frac{jb}{ic}\right)^2\right]^2}B_j}{1+\frac{i\pi c}{b}\left(\coth\frac{i\pi c}{b}-\tanh\frac{i\pi c}{b}\right)} \quad \ldots\ldots (37)$$

$$A_{ij} = -\left[\frac{ES_\infty\, 4\,(-1)^{i+j}\, i^2 j^2}{(1-\mu)\,\pi^2\left(i^2\frac{c}{b}+j^2\frac{b}{c}\right)^2}\right] \sum_{n=1}^{\infty}\sum_{m=1}^{\infty}\frac{\left(\beta_n^2\frac{c}{b}+\beta_m^2\frac{b}{c}\right)e^{-T\beta_n^2}\, e^{-T_c\beta_m^2}\, H_n H_m}{\left(1-\frac{i^2\pi^2}{\beta_n^2}\right)\left(1-\frac{j^2\pi^2}{\beta_m^2}\right)} \quad (38)$$

$$A_o = -\frac{ES_\infty}{1-\mu}\sum_{n=1}^{\infty}\sum_{m=1}^{\infty}\left(\beta_n^2\frac{c}{b}+\beta_m^2\frac{b}{c}\right)e^{-T\beta_n^2}\, e^{-T_c\beta_m^2}\, H_n H_m \ldots (39)$$

The above equations, together with Equation 31 for σ_z, constitute a complete solution on the basis of the given assumptions.

In general, if both the parameters T and T_c are equal to or greater than 0.1, the series given above converge very rapidly so that only a few terms need be taken for a good approximation. The example given below will demonstrate the use of the above equations.

Example: *Stress at the middle of one side of a square prism for which B equals 5 and at a time for which T equals 0.1.* If only one term in each series is used, the following values are obtained: From Equations 38 and 39 and Table 1.

$$A_o = -\frac{ES_\infty}{1-\mu}\,(\beta_1^2+\beta_1^2)\, e^{-0.1\,\beta_1^2}\, e^{-0.1\,\beta_1^2}\, H_1^2$$

$$A_o = -\frac{ES_\infty}{1-\mu}(1.3138^2+1.3138^2)\,0.8415^2\times 0.9130^2$$

$$A_o = -2.0377\,\frac{ES_\infty}{1-\mu}$$

$$A_{11} = -\frac{ES_\infty}{(1-\mu)\,\pi^2}\,\frac{(1.3138^2+1.3138^2)\,0.8415^2\times 0.9130^2}{\left(1-\frac{\pi^2}{1.3138^2}\right)^2}$$

$$A_{11} = -0.0093 \frac{ES_\infty}{1-\mu}$$

When these values are substituted into Equation 36, the result is

$$B_1 = \frac{\left(-\frac{2.0376}{\pi^2} + 0.0093\right)\frac{ES_\infty}{1-\mu} - \frac{tanh\ \pi}{\pi} C_1}{1 + \pi\ (coth\ \pi - tanh\ \pi)}$$

or since $C_1 = B_1$ (square prism),

$$C_1 = B_1 = -0.1472 \frac{ES_\infty}{1-\mu}$$

When the above values and $z = c$, $y = 0$ are substituted into Equation 33, the result is

$$\sigma_y \Big|_{\substack{z=c \\ y=0}} = \left[\frac{2.0377}{6} + 0.0093 - 0.1472 \times 0.3583 + 0.1472 \times 0.9773\right]\frac{ES_\infty}{1-\mu}$$

$$\sigma_y \Big|_{\substack{z=c \\ y=0}} = \left[+0.3396 + 0.0093 - 0.0527 + 0.1439\right]\frac{ES_\infty}{1-\mu} = 0.4401 \frac{ES_\infty}{1-\mu}$$

When the summation of each series is carried to two terms, the result is $0.4214 \frac{ES_\infty}{1-\mu}$ and when the summation is carried to four terms in each series the result is $0.4221 \frac{ES_\infty}{1-\mu}$ for this stress.

The above shows that the series converge very rapidly for this example.

The stresses σ_z and τ_{yz} are obviously zero at the point under consideration.

From Equations 31, 5b, and 22b, Tables 2 and 3, and the above result for σ_y,

$$\sigma_x \Big|_{\substack{z=c \\ y=0}} = \left[\frac{\mu}{1-\mu} 0.4221 + (0.0221 + 0.6913 - 0.0221 \times 0.6913) - (0.2186 + 0.2186 - 0.2186 \times 0.2186)\right] ES_\infty$$

$$\sigma_x \Big|_{\substack{z=c \\ y=0}} = \left[\frac{\mu}{1-\mu} 0.4221 + 0.3087\right] ES_\infty$$

For all values of Poisson's ratio μ less than 0.212, the above expressions will give larger values for σ_y than for σ_x. This fact is of interest because,

in the similar problem of a long cylinder drying or cooling from the curved surface the two corresponding stresses are equal regardless of Poisson's ratio[13].

OTHER METHODS

A paper of this kind would not be complete without calling attention to the possibilities of using a *difference equation* instead of Equation 1. The *difference equation* that would replace Equation 1 would be a statement of the relation between the values of S at the point (x, y, z) at the time t and what the values of S were at this point and at points distant $\pm\Delta x$, $\pm\Delta y$ and $\pm\Delta z$ in the x-, y-, and z-directions, Δt units of time previously. In the limit as Δx, Δy, Δz and Δt are made smaller, the *difference equation* becomes Equation 1. Although methods based upon the substitution of a *difference equation* for the differential equation usually necessitate considerable work to obtain the desired results, they have the advantage of being applicable to any shape of body, any assumption in regard to boundary conditions, or any variation in the coefficient of diffusivity.

Methods of procedure for solving the *difference equation*, usually for the analogous problem of heat-flow, may be found in the literature*.

After a solution for the distribution of shrinkage tendency at a given time has been obtained to the desired accuracy, there still remains the problem of solving for stresses. This may be done by graphical analysis, as illustrated by Buchanan and Schroeder[16] if the problem is one-dimensional or by more elaborate means if the problem is two- or three-dimensional.†

SUMMARY OF THE THEORETICAL DEVELOPMENT

The theoretical equations for the flow of heat are used for the unrestrained shrinkage of concrete. Unrestrained shrinkage is defined as the unit linear contraction that would occur in an element if it were unrestrained by neighboring elements. This is not the same as unit shortening (commonly called shrinkage) of a so-called unrestrained specimen. Having developed expressions for unrestrained shrinkage for each of three different conditions (Cases I, II and III), equations for shortening, warping, and stresses were derived.

The equations for two conditions—slab or beam drying from one face only, Case I, and slab or beam drying from two opposite faces, Case II—were found to be very much alike in form. However, the equation for shrinkage stress in the slab or beam drying from one surface contains an additional term, $\left(6 - 12\,\frac{y}{b}\right)\frac{2Ebv_{max}}{3l^2}$, that is not in the corresponding

*See, for example, References 14 and 15.
†See, for example, References 17, 18, and 19

equation for a slab or beam drying from two opposite surfaces. Computations show that the equation with the added term gives much less stress for the same size of body and the same period of drying. Compare Tables 6 and 5.

Equations are given for all the stresses in a prism drying from all four surfaces, the third condition treated. These equations, though rather complicated in appearance, can be readily evaluated if the desired accuracy is such that only a few terms in each series need be considered. For a rather rough approximation of the stress that is usually the most important in this third condition, the comparatively simple equation $\sigma_x = E(S - S_{av})$ is recommended.

Tables and curves are given from which the theoretical shrinkages, stresses, etc., may be obtained, at any point in the specimen after any period of drying, for various values of the physical properties, diffusivity, surface factor, ultimate shrinkage, and dimensions of the specimen.

Examples are given showing how numerical values may be computed from the equations and how the tables and curves may be used.

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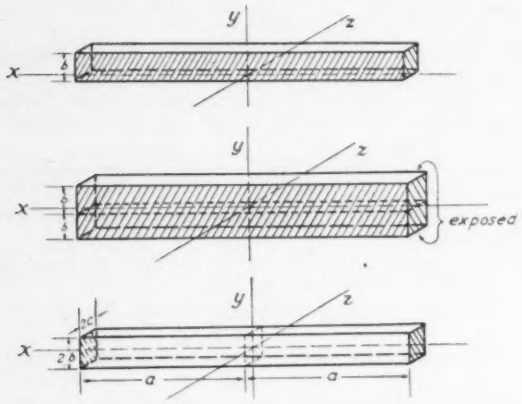

Fig. 1—Illustrations of the conditions treated. Shading indicates sealed surfaces. Case I—Beam (or slab) drying from one face only. Case II—Beam (or slab) drying from two opposite faces. Case III—Prism drying from four faces. The ends of the prism at $x = \pm a$ are sealed.

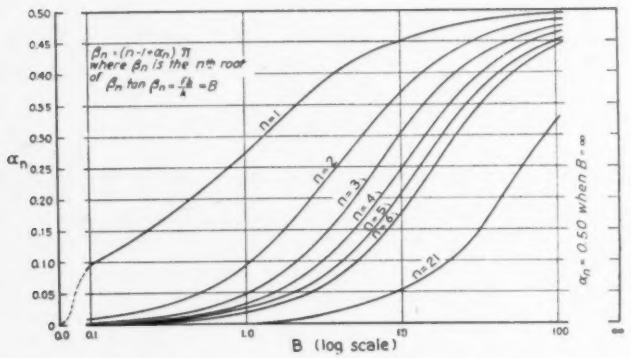

Fig. 2—Curves for the determination of β_n

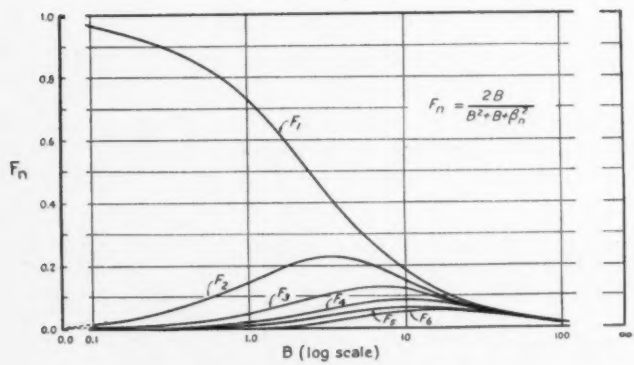

Fig. 3—Relationship between F_n and B

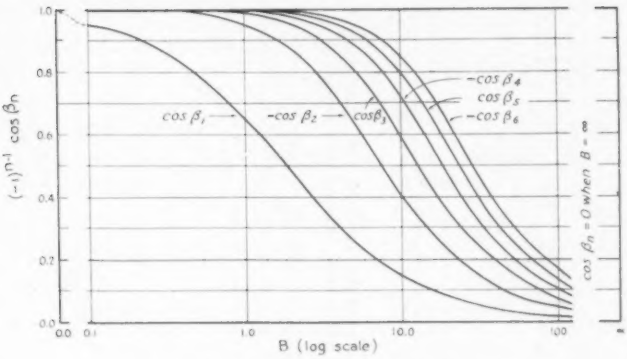

Fig. 4—Relationship between cos β_n and B

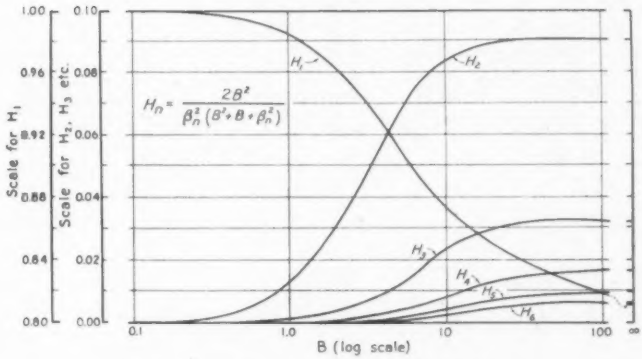

Fig. 5—Relationship between H_n and B

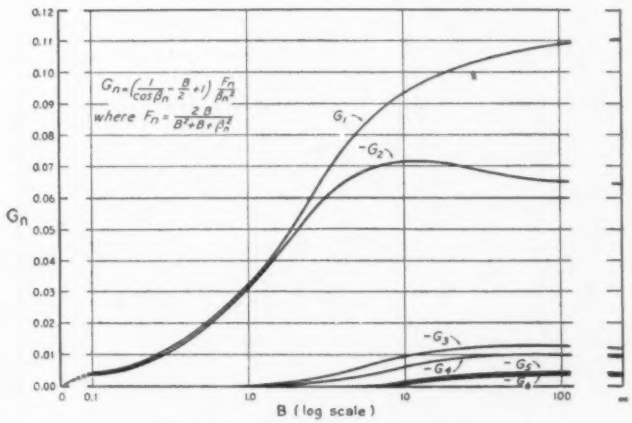

Fig. 6—Relationship between G_n and B

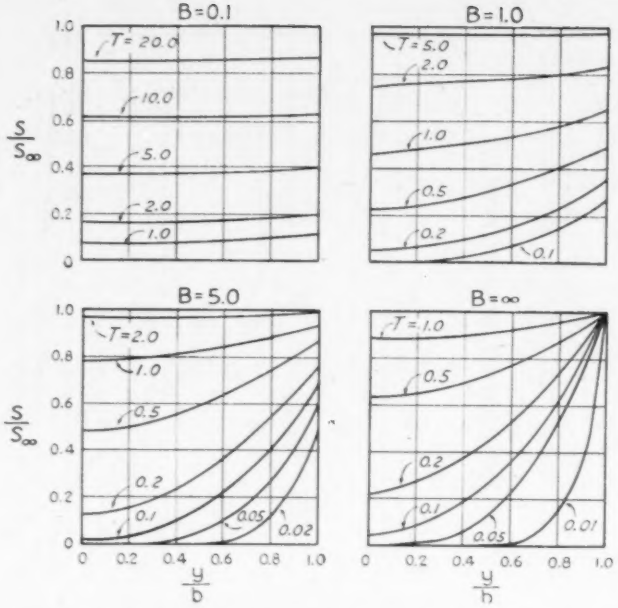

Fig. 7—Distribution of shrinkage for various values of B and T

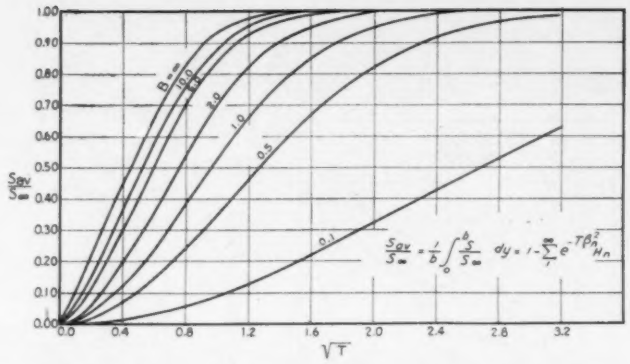

Fig. 8—Theoretical curves showing the ratio of unit shortening to final shortening vs. the square root of the time parameter

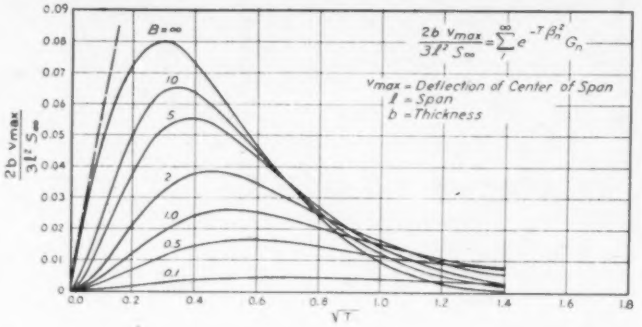

Fig. 9—Theoretical curves showing warping vs. square root of the time parameter for a prism drying from one side

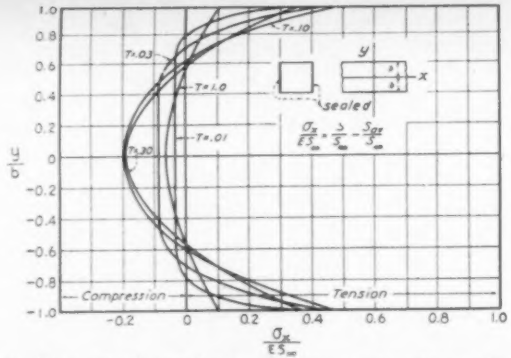

Fig. 10—Theoretical distribution of stresses at various values of T for Case II (or Case I where warping is prevented). $B = 5.0$

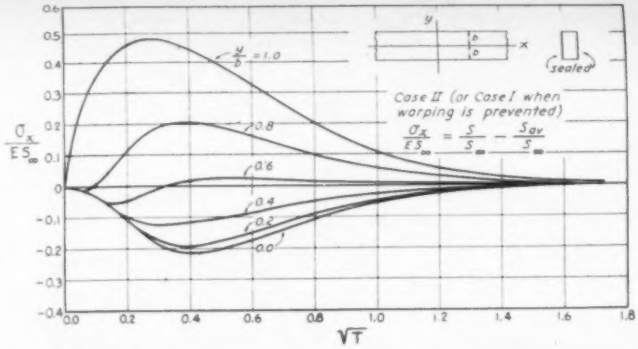

Fig. 12—Stress vs. square root of T at various distances from central plane of prisms drying from two opposite sides. $B = 5.0$

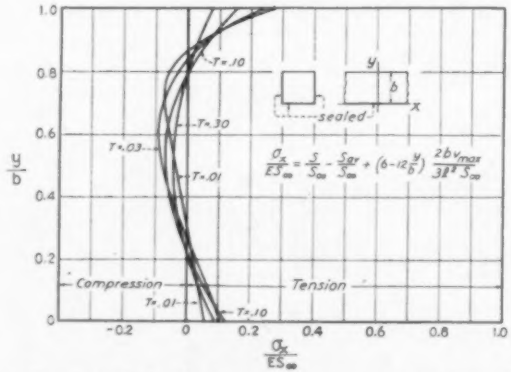

Fig. 11—Theoretical distribution of stress at various values of T for Case I. $B = 5.0$

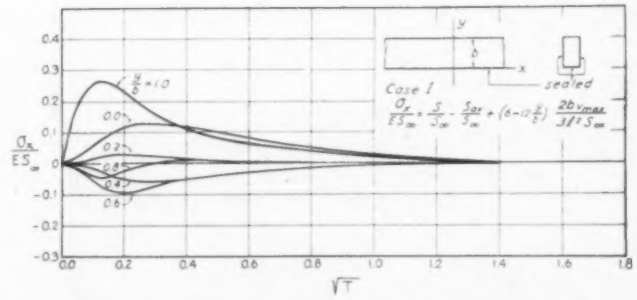

Fig. 13—Stress vs. square root of T at various values of y/b for an unrestrained prism drying from only one surface. $B = 5.0$

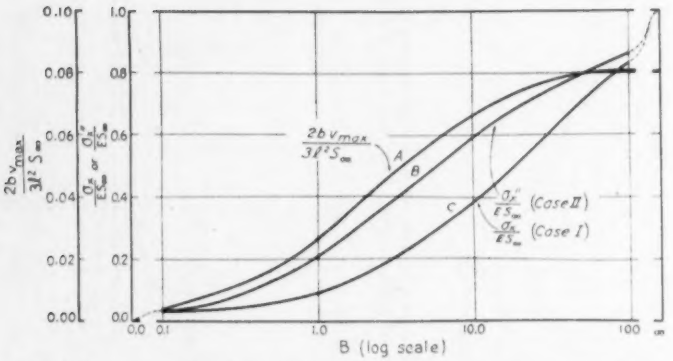

Fig. 14—Maximum stress (maximum value of stress at exposed surface) and maximum warping vs. the parameter B

TABLE 1—β_n

$\beta_n \tan \beta_n = B$

B	β_1	β_2	β_3	β_4	β_5	β_6
0.0	0	π	2π	3π	4π	5π
0.1	0.311053	3.1731	6.2991	9.4354	12.574	15.715
0.5	0.653271	3.2923	6.3615	9.4774	12.606	15.740
1.0	0.860334	3.4256	6.4372	9.5292	12.645	15.771
2.0	1.076874	3.6436	6.5783	9.6296	12.722	15.834
5.0	1.31384	4.0338	6.9097	9.8927	12.935	16.010
10.0	1.42886	4.3058	7.2281	10.200	13.213	16.260
100.0	1.55525	4.6656	7.7760	10.887	13.998	17.109
∞	$\pi/2$	$3\pi/2$	$5\pi/2$	$7\pi/2$	$9\pi/2$	$11\pi/2$

TABLE 2—RATIO OF SHRINKAGE (OR SWELLING) TO ULTIMATE SHRINKAGE (OR SWELLING)

$$\frac{S}{S_\infty} = 1 - \Sigma e^{-T\beta_n^2} F_n \frac{\cos \beta_n \frac{y}{b}}{\cos \beta_n}$$

T	$\frac{y}{b}=0$	$\frac{y}{b}=0.2$	$\frac{y}{b}=0.4$	$\frac{y}{b}=0.6$	$\frac{y}{b}=0.8$	$\frac{y}{b}=1.0$	$\frac{y}{b}=0$	$\frac{y}{b}=0.2$	$\frac{y}{b}=0.4$	$\frac{y}{b}=0.6$	$\frac{y}{b}=0.8$	$\frac{y}{b}=1.0$
	$B=0.1$						$B=1.0$					
0.0	0	0	0	0	0	0	0	0	0	0	0	0
0.005	.0000	.0000	.0000	.0000	.0002	.0080	.0000	.0000	.0000	.0000	.0007	.0750
0.010	.0000	.0000	.0000	.0000	.0009	.0112	.0000	.0000	.0000	.0002	.0093	.1035
0.015	.0000	.0000	.0000	.0001	.0021	.0135	.0000	.0000	.0000	.0012	.0196	.1238
0.020	.0000	.0000	.0000	.0003	.0033	.0158	.0000	.0000	.0005	.0033	.0305	.1416
0.030	.0000	.0000	.0001	.0011	.0057	.0192	.0000	.0001	.0013	.0095	.0511	.1690
0.040	.0000	.0000	.0003	.0020	.0080	.0222	.0001	.0003	.0033	.0182	.0700	.1910
0.050	.0000	.0001	.0007	.0030	.0100	.0247	.0003	.0010	.0065	.0274	.0870	.2096
0.075	.0002	.0005	.0020	.0060	.0146	.0302	.0021	.0050	.0178	.0516	.1236	.2471
0.10	.0008	.0014	.0037	.0089	.0188	.0347	.0068	.0121	.0317	.0749	.1538	.2764
0.15	.0029	.0040	.0077	.0146	.0260	.0423	.0244	.0333	.0623	.1168	.2025	.3218
0.20	.0060	.0074	.0119	.0198	.0319	.0486	.0499	.0609	.0950	.1545	.2418	.3570
0.30	.0139	.0156	.0209	.0298	0.426	.0594	.1036	.1164	.1534	.2153	.3012	.4091
0.40	.0228	.0246	.0301	.0393	.0523	.0690	.1691	.1811	.2169	.2757	.3560	.4559
0.50	.0320	.0339	.0394	.0487	.0616	.0782	.2274	.2387	.2725	.3276	.4026	.4954
0.75	.0550	.0569	.0623	.0714	.0841	.1004	.3575	.3670	.3953	.4413	.5038	.5810
1.0	.0776	.0794	.0847	.0937	.1060	.1219	.4662	.4741	.4975	.5358	.5878	.6518
1.5	.1213	.1230	.1281	.1364	.1484	.1634	.6312	.6367	.6528	.6793	.7152	.7595
2.0	.1627	.1643	.1691	.1772	.1885	.2028	.7453	.7490	.7603	.7785	.8033	.8339
3.0	.2399	.2414	.2458	.2531	.2634	.2764	.8785	.8803	.8856	.8943	.9061	.9207
4.0	.3100	.3114	.3154	.3220	.3313	.3432	.9420	.9429	.9454	.9496	.9552	.9622
5.0	.3736	.3748	.3785	.3845	.3929	.4037	.9724	.9728	.9740	.9760	.9786	.9820
7.5	.5082	.5092	.5120	.5167	.5234	.5318						
10.0	.6240	.6247	.6269	.6306	.6356	.6421						
15.0	.7620	.7625	.7638	.7661	.7693	.7734						
20.0	.8533	.8535	.8544	.8558	.8578	.8603						
	$B=5.0$						$B=\infty$					
0.0	0	0	0	0	0	0	0	0	0	0	0	0
0.001	.0000	.0000	.0000	.0000	.0000	.1559	.0000	.0000	.0000	.0000	.0000	1.000
0.002	.0000	.0000	.0000	.0000	.0001	.2095	.0000	.0000	.0000	.0000	.0016	1.000
0.005	.0000	.0000	.0000	.0000	.0072	.3019	.0000	.0000	.0000	.0001	.0455	1.000
0.010	.0000	.0000	.0000	.0008	.0390	.3843	.0000	.0000	.0002	.0060	.1541	1.000
0.015	.0000	.0000	.0000	.0048	.0770	.4378	.0000	.0000	.0005	.0209	.2483	1.000
0.020	.0000	.0003	.0004	.0126	.1132	.4769	.0000	.0001	.0027	.0455	.3173	1.000
0.030	.0001	.0010	.0043	.0353	.1762	.5332	.0001	.0010	.0144	.1025	.4142	1.000
0.040	.0006	.0017	.0117	.0634	.2291	.5724	.0008	.0047	.0339	.1573	.4795	1.000
0.050	.0020	.0042	.0225	.0918	.2723	.6026	.0031	.0115	.0578	.2059	.5271	1.000
0.075	.0070	.0175	.0572	.1579	.3544	.6557	.0198	.0407	.1216	.3017	.6056	1.000
0.10	.0221	.0390	.0956	.2142	.4134	.6913	.0410	.0717	.1735	.3657	.6517	1.000
0.15	.0725	.0962	.1706	.3025	.4947	.7379	.1358	.1726	.2839	.4687	.7160	1.000
0.20	.1346	.1613	.2398	.3709	.5506	.7685	.2276	.2637	.3710	.5384	.7558	1.000
0.30	.2637	.2882	.3607	.4771	.6304	.8106	.3933	.4227	.5085	.6425	.8119	1.000
0.40	.3788	.3999	.4621	.5613	.6907	.8417	.5256	.5487	.6161	.7210	.8533	1.000
0.50	.4769	.4948	.5474	.6311	.7401	.8670	.6294	.6475	.7001	.7821	.8855	1.000
0.75	.6602	.6718	.7061	.7604	.8312	.9136	.8000	.8098	.8382	.8824	.9382	1.000
1.0	.7793	.7869	.8091	.8444	.8904	.9439	.8920	.8972	.9126	.9365	.9666	1.000
1.5	.9069	.9101	.9145	.9344	.9538	.9763	.9686	.9700	.9746	.9815	.9903	1.000
2.0	.9607	.9621	.9660	.9723	.9805	.9900	.9908	.9913	.9926	.9946	.9972	1.000
3.0	.9930	.9933	.9940	.9951	.9965	.9982	.9992	.9993	.9994	.9995	.9998	1.000
4.0	.9988	.9988	.9990	.9991	.9994	.9997						

TABLE 3—UNIT SHORTENING—AVERAGE UNIT SHRINKAGE $\left(\frac{S_{av}}{S_\infty}\right)$

$$\frac{S_{av}}{S_\infty} = 1 - \sum_{1}^{\infty} e^{-T\beta^2_n} H_n$$

T	$B = 0.1$	$B = 0.5$	$B = 1.0$	$B = 2.0$	$B = 5.0$	$B = 10.0$	$B = \infty$
0.001	0.0001	0.0005	0.0010	0.0019	0.0045	0.0080	0.0357
0.002	0.0002	0.0010	0.0019	0.0037	0.0085	0.0147	0.0506
0.003	0.0003	0.0015	0.0029	0.0055	0.0124	0.0206	0.0619
0.005	0.0005	0.0024	0.0047	0.0090	0.0196	0.0323	0.0800
0.010	0.0010	0.0048	0.0093	0.0173	0.0370	0.0556	0.1129
0.015	0.0015	0.0072	0.0137	0.0252	0.0515	0.0755	0.1383
0.02	0.0020	0.0095	0.0181	0.0317	0.0649	0.0932	0.1596
0.03	0.0030	0.0141	0.0265	0.0473	0.0891	0.1242	0.1954
0.04	0.0039	0.0185	0.0347	0.0611	0.1115	0.1512	0.2257
0.05	0.0049	0.0231	0.0426	0.0742	0.1319	0.1753	0.2523
0.075	0.0074	0.0340	0.0620	0.1050	0.1779	0.2282	0.3090
0.10	0.0098	0.0446	0.0803	0.1336	0.2186	0.2739	0.3506
0.15	0.0146	0.0653	0.1154	0.1860	0.2895	0.3510	0.4370
0.20	0.0193	0.0854	0.1489	0.2327	0.3510	0.4167	0.5041
0.30	0.0288	0.1239	0.2064	0.3190	0.4555	0.5258	0.6133
0.40	0.0382	0.1606	0.2666	0.3939	0.5422	0.6136	0.6979
0.50	0.0474	0.1957	0.3188	0.4604	0.6148	0.6849	0.7641
0.75	0.0702	0.2771	0.4339	0.5962	0.7498	0.8109	0.8726
1.0	0.0924	0.3502	0.5296	0.6979	0.8375	0.8865	0.9313
1.5	0.1353	0.4751	0.6751	0.8306	0.9315	0.9591	0.9800
2.0	0.1761	0.5767	0.7756	0.9052	0.9711	0.9857	0.9942
3.0	0.2521	0.7233	0.8930	0.9625	0.9949	0.9981	0.9995
4.0	0.3211	0.8182	0.9489	0.9907	0.9991	0.9998	1.0000
5.0	0.3837	0.8821	0.9756	0.9971	0.9998	1.0000	
7.5	0.5161	0.9594	0.9962	0.9998	1.0000		
10.0	0.6301	0.9860	0.9994	1.0000			
15.0	0.7658	0.9983	1.0000				
20.0	0.8556	0.9998					

TABLE 4—WARPING $\left(\frac{2b\,v_{max}}{3l^2 S_\infty} \text{ in thousandths}\right)$

$$\frac{2b\,v_{max}}{3l^2\,S_\infty} = \sum_{1}^{\infty} e^{-T\beta^2_n} G_n$$

T	$B = 0.1$	$B = 0.5$	$B = 1.0$	$B = 2.0$	$B = 5.0$	$B = 10.0$	$B = \infty$
0	0	0	0	0	0	0	0
0.0010	0.048	0.235	0.46	0.91	2.13	3.82	16.83
0.0015	0.071	0.348	0.68	1.33	3.08	5.43	19.79
0.0020	0.093	0.458	0.90	1.73	3.99	6.89	23.22
0.0030	0.137	0.674	1.32	2.54	5.73	9.61	27.90
0.0040	0.180	0.883	1.72	3.31	7.23	12.03	31.66
0.0050	0.222	1.008	2.12	4.03	8.69	14.27	34.89
0.0075	0.324	1.579	3.06	5.74	13.68	19.16	41.36
0.010	0.421	2.045	3.94	7.35	15.18	23.35	46.42
0.015	0.606	2.920	5.59	10.25	20.45	30.38	54.12
0.020	0.778	3.735	7.08	12.92	24.97	35.92	59.78
0.030	1.092	5.415	9.74	17.34	32.63	44.71	67.67
0.040	1.375	6.518	12.05	21.07	38.32	50.93	72.80
0.050	1.628	7.832	14.03	24.21	42.70	55.33	76.15
0.075	2.161	10.138	17.96	30.08	49.98	62.22	79.76
0.10	2.568	11.846	20.75	33.87	53.81	65.16	80.40
0.15	3.126	14.017	23.96	37.56	55.70	64.50	74.18
0.20	3.457	15.128	25.29	38.32	53.89	60.56	66.85
0.30	3.757	15.773	25.61	36.28	47.03	50.52	52.74
0.40	3.843	15.546	24.14	32.88	39.90	41.38	41.27
0.50	3.851	15.043	22.59	29.43	33.64	33.77	32.25
0.75	3.782	13.588	18.85	22.07	21.86	20.27	17.40
1.0	3.694	12.217	15.67	16.52	14.20	12.17	9.39
1.5	3.519	9.869	10.82	9.25	5.99	4.38	2.74
2.0	3.353	7.959	7.47	5.18	2.53	1.58	0.80
3.0	3.044	5.203	3.57	2.05	0.45	0.20	0.07
4.0	2.763	3.417	1.70	0.51	0.08	0.03	0.01
5.0	2.508	2.216	0.81	0.16	0.01	0.00	0.00
7.5	1.969	0.763	0.13	0.01	0.00		
10.0	1.506	0.262	0.02	0.00			
15.0	0.953	0.031	0.00				
20.0	0.587	0.004					

TABLE 5—RATIO OF STRESS σ''_z IN AN UNRESTRAINED BEAM DRYING FROM TWO OPPOSITE SIDES (OR IN A BEAM DRYING FROM ONLY ONE SIDE AND RESTRAINED AGAINST WARPING) TO THE ULTIMATE STRESS FOR COMPLETE RESTRAINT ES_∞

$$\frac{\sigma''_z}{ES_\infty} = \frac{S}{S_\infty} - \frac{S_{av}}{S_\infty}$$

T	$\frac{y}{b}=0$	$\frac{y}{b}=0.2$	$\frac{y}{b}=0.4$	$\frac{y}{b}=0.6$	$\frac{y}{b}=0.8$	$\frac{y}{b}=1.0$	$\frac{y}{b}=0$	$\frac{y}{b}=0.2$	$\frac{y}{b}=0.4$	$\frac{y}{b}=0.6$	$\frac{y}{b}=0.8$	$\frac{y}{b}=1.0$
	$B=0.1$						$B=1.0$					
0.0	0	0	0	0	0	0	0	0	0	0	0	0
0.005	-.0005	-.0005	-.0005	-.0005	-.0003	.0075	-.0047	-.0047	-.0047	-.0047	-.0040	.0703
0.010	-.0010	-.0010	-.0010	-.0010	-.0001	.0102	-.0093	-.0093	-.0093	-.0091	.0000	.0942
0.015	-.0015	-.0015	-.0015	-.0014	.0006	.0120	-.0137	-.0137	-.0137	-.0125	.0059	.1101
0.020	-.0020	-.0020	-.0020	-.0017	.0013	.0138	-.0181	-.0181	-.0176	-.0148	.0124	.1235
0.030	-.0030	-.0030	-.0029	-.0019	.0027	.0162	-.0265	-.0264	-.0252	-.0170	.0246	.1425
0.040	-.0039	-.0039	-.0036	-.0019	.0041	.0183	-.0346	-.0344	-.0314	-.0165	.0353	.1563
0.050	-.0049	-.0048	-.0042	-.0019	.0051	.0198	-.0423	-.0416	-.0361	-.0152	.0444	.1670
0.075	-.0072	-.0069	-.0054	-.0014	.0072	.0228	-.0599	-.0570	-.0442	-.0104	.0616	.1851
0.10	-.0090	-.0084	-.0061	-.0009	.0090	.0249	-.0735	-.0682	-.0486	-.0054	.0735	.1961
0.15	-.0117	-.0106	-.0069	.0000	.0114	.0277	-.0910	-.0821	-.0531	.0014	.0871	.2064
0.20	-.0133	-.0119	-.0074	.0005	.0126	.0293	-.0990	-.0880	-.0539	.0056	.0929	.2081
0.30	-.0149	-.0132	-.0079	.0010	.0138	.0306	-.1028	-.0900	-.0530	.0089	.0948	.2027
0.40	-.0154	-.0136	-.0081	.0011	.0141	.0308	-.0975	-.0855	-.0497	.0091	.0894	.1893
0.50	-.0154	-.0135	-.0080	.0013	.0142	.0311	-.0914	-.0801	-.0463	.0088	.0838	.1766
0.75	-.0152	-.0133	-.0079	.0013	.0139	.0302	-.0764	-.0669	-.0386	.0074	.0699	.1471
1.0	-.0148	-.0130	-.0077	.0012	.0136	.0295	-.0634	-.0555	-.0321	.0062	.0582	.1222
1.5	-.0140	-.0123	-.0072	.0011	.0131	.0281	-.0439	-.0384	-.0223	.0042	.0401	.0844
2.0	-.0134	-.0118	-.0070	.0011	.0124	.0267	-.0303	-.0266	-.0153	.0029	.0277	.0583
3.0	-.0122	-.0107	-.0063	.0010	.0113	.0243	-.0145	-.0127	-.0074	.0013	.0131	.0277
4.0	-.0111	-.0097	-.0057	.0009	.0102	.0221	-.0069	-.0060	-.0035	.0007	.0063	.0133
5.0	-.0101	-.0089	-.0052	.0008	.0092	.0200	-.0032	-.0028	-.0016	.0004	.0030	.0064
7.5	-.0079	-.0069	-.0041	.0006	.0073	.0157						
10.0	-.0061	-.0055	-.0033	.0005	.0055	.0120						
15.0	-.0038	-.0033	-.0020	.0003	.0035	.0076						
20.0	-.0023	-.0021	-.0012	.0002	.0022	.0047						
	$B=5.0$						$B=\infty$					
0.0	0	0	0	0	0	0	0	0	0	0	0	0
0.001	-.0045	-.0045	-.0045	-.0045	-.0045	.1514	-.0357	-.0357	-.0357	-.0357	-.0357	.9743
0.002	-.0085	-.0085	-.0085	-.0085	-.0084	.2010	-.0506	-.0506	-.0506	-.0506	-.0490	.9494
0.005	-.0196	-.0196	-.0196	-.0196	-.0124	.2823	-.0800	-.0800	-.0800	-.0799	-.0345	.9200
0.010	-.0370	-.0370	-.0370	-.0362	.0020	.3473	-.1129	-.1129	-.1127	-.1069	.0412	.8871
0.015	-.0515	-.0515	-.0515	-.0467	.0255	.3863	-.1383	-.1383	-.1378	-.1174	.1100	.8617
0.020	-.0649	-.0646	-.0645	-.0523	.0483	.4120	-.1596	-.1595	-.1569	-.1141	.1577	.8404
0.030	-.0890	-.0881	-.0848	-.0538	.0871	.4441	-.1953	-.1944	-.1810	-.0929	.2188	.8046
0.040	-.1109	-.1098	-.0998	-.0481	.1176	.4609	-.2249	-.2210	-.1918	-.0684	.2538	.7743
0.050	-.1299	-.1277	-.1094	-.0401	.1404	.4707	-.2492	-.2408	-.1945	-.0464	.2748	.7477
0.075	-.1709	-.1604	-.1207	-.0200	.1765	.4778	-.2892	-.2683	-.1874	-.0073	.2966	.6910
0.10	-.1965	-.1796	-.1230	-.0044	.1948	.4727	-.3096	-.2789	-.1771	.0151	.3011	.6494
0.15	-.2169	-.1969	-.1189	-.0130	.2052	.4484	-.3012	-.2644	-.1531	.0317	.2790	.5630
0.20	-.2164	-.1897	-.1112	-.0199	.1996	.4175	-.2765	-.2404	-.1331	.0343	.2517	.4959
0.30	-.1918	-.1671	-.0948	.0216	.1749	.3551	-.2200	-.1906	-.1048	.0292	.1986	.3867
0.40	-.1634	-.1422	-.0801	.0191	.1485	.2995	-.1723	-.1492	-.0818	.0231	.1554	.3021
0.50	-.1379	-.1193	-.0674	.0163	.1253	.2522	-.1347	-.1166	-.0640	.0180	.1214	.2359
0.75	-.0896	-.0779	-.0437	.0106	.0814	.1638	-.0727	-.0629	-.0345	.0097	.0655	.1273
1.0	-.0582	-.0506	-.0284	.0069	.0529	.1064	-.0393	-.0341	-.0187	.0052	.0352	.0687
1.5	-.0246	-.0214	-.0170	.0029	.0223	.0448	-.0114	-.0100	-.0054	.0015	.0103	.0200
2.0	-.0104	-.0090	-.0051	.0012	.0094	.0189	-.0034	-.0029	-.0016	.0004	.0030	.0058
3.0	-.0019	-.0016	-.0009	.0002	.0016	.0033						

TABLE 6—RATIO OF STRESS σ_x IN AN UNRESTRAINED BEAM DRYING FROM ONLY ONE SIDE TO ULTIMATE STRESS FOR COMPLETE RESTRAINT ES_∞

$$\frac{\sigma_x}{ES_\infty} = \frac{S}{S_\infty} - \frac{S_{av}}{S_\infty} + (6 - 12\,\frac{y}{b})\,\frac{2bv_{max}}{3P^2S_\infty}$$

T	$\frac{y}{b}=0$	$\frac{y}{b}=0.2$	$\frac{y}{b}=0.4$	$\frac{y}{b}=0.6$	$\frac{y}{b}=0.8$	$\frac{y}{b}=1.0$	$\frac{y}{b}=0$	$\frac{y}{b}=0.2$	$\frac{y}{b}=0.4$	$\frac{y}{b}=0.6$	$\frac{y}{b}=0.8$	$\frac{y}{b}=1.0$
	$B = 0.1$						$B = 1.0$					
0.0	0	0	0	0	0	0	0	0	0	0	0	0
0.005	.0008	.0003	-.0002	-.0008	-.0011	.0062	.0080	.0029	-.0022	-.0072	-.0116	.0576
0.010	.0015	.0005	-.0005	-.0015	-.0016	.0078	.0143	.0049	-.0046	-.0138	-.0142	.0706
0.015	.0021	.0007	-.0008	-.0021	-.0016	.0084	.0198	.0064	-.0070	-.0192	-.0142	.0766
0.020	.0027	.0008	-.0011	-.0026	-.0015	.0091	.0243	.0074	-.0091	-.0233	-.0131	.0811
0.030	.0036	.0009	-.0016	-.0032	-.0012	.0096	.0319	.0087	-.0135	-.0287	-.0105	.0841
0.040	.0044	.0011	-.0019	-.0036	-.0009	.0100	.0377	.0090	-.0169	-.0310	-.0081	.0840
0.050	.0048	.0011	-.0022	-.0039	-.0008	.0100	.0419	.0089	-.0193	-.0320	-.0061	.0828
0.075	.0058	.0009	-.0028	-.0040	-.0006	.0098	.0479	.0077	-.0226	-.0320	-.0031	.0773
0.10	.0064	.0008	-.0031	-.0040	-.0002	.0095	.0510	.0065	-.0237	-.0303	-.0012	.0716
0.15	.0071	.0007	-.0031	-.0038	.0001	.0089	.0528	.0042	.0243	-.0274	.0008	.0626
0.20	.0074	.0005	-.0033	-.0036	.0002	.0085	.0527	.0030	-.0236	-.0247	.0019	.0564
0.30	.0076	.0003	-.0034	-.0035	.0003	.0080	.0509	.0022	-.0223	-.0218	.0026	.0490
0.40	.0077	.0002	-.0035	-.0035	.0003	.0077	.0473	.0014	-.0207	-.0199	.0025	.0445
0.50	.0077	.0004	-.0034	-.0033	.0002	.0077	.0441	.0012	-.0192	-.0183	.0025	.0318
0.75	.0075	.0003	-.0034	-.0032	.0003	.0075	.0367	.0010	-.0160	-.0152	.0020	.0340
1.0	.0074	.0003	-.0033	-.0032	.0003	.0073	.0306	.0009	-.0133	-.0126	.0018	.0282
1.5	.0071	.0004	-.0030	-.0031	.0004	.0070	.0210	.0006	-.0093	-.0088	.0011	.0195
2.0	.0067	.0003	-.0030	-.0029	.0003	.0066	.0145	.0003	-.0063	-.0061	.0008	.0135
3.0	.0061	.0003	-.0026	-.0027	.0003	.0060	.0069	.0002	-.0031	-.0030	.0002	.0063
4.0	.0055	.0002	-.0025	-.0024	.0003	.0055	.0033	.0001	-.0015	-.0013	.0002	.0031
5.0	.0049	.0001	-.0022	-.0022	.0002	.0050	.0017	.0001	-.0006	-.0006	.0001	.0015
7.5	.0039	.0001	-.0017	-.0020	.0002	.0039						
10.0	.0030	.0001	-.0014	-.0014	.0001	.0030						
15.0	.0019	.0001	-.0009	-.0008	.0001	.0019						
20.0	.0012	.0000	-.0005	-.0005	.0001	.0012						
	$B = 5.0$						$B = \infty$					
0.0	0	0	0	0	0	0	0	0	0	0	0	0
0.001	.0083	.0032	-.0019	-.0071	-.0122	.1386	.0653	.0249	-.0155	-.0559	-.0963	.8733
0.002	.0154	.0059	-.0037	-.0133	-.0288	.1771	.0887	.0330	-.0227	-.0785	-.1326	.8101
0.005	.0325	.0117	-.0092	-.0300	-.0437	.2302	.1293	.0456	-.0381	-.1218	-.1601	.7107
0.010	.0541	.0176	-.0188	-.0544	-.0526	.2562	.1656	.0542	-.0570	-.1626	-.1259	.6086
0.015	.0712	.0221	-.0270	-.0712	-.0481	.2636	.1864	.0565	-.0729	-.1823	-.0848	.5370
0.020	.0849	.0250	-.0345	-.0822	-.0413	.2622	.1991	.0557	-.0852	-.1858	-.0575	.4817
0.030	.1068	.0294	-.0456	-.0930	-.0304	.2483	.2109	.0493	-.0998	-.1741	-.0249	.3984
0.040	.1190	.0282	-.0538	-.0941	-.0204	.2310	.2119	.0411	-.1044	-.1558	-.0083	.3375
0.050	.1263	.0260	-.0582	-.0913	-.0133	.2145	.2080	.0335	-.1031	-.1378	.0005	.2905
0.075	.1290	.0195	-.0607	-.0800	-.0034	.1779	.1896	.0190	-.0916	-.1031	.0093	.2122
0.10	.1264	.0141	-.0584	-.0690	.0011	.1498	.1728	.0105	-.0806	-.0814	.0117	.1670
0.15	.1173	.0072	-.0521	-.0538	.0047	.1142	.1439	.0027	-.0641	-.0573	.0119	.1179
0.20	.1069	.0043	-.0465	-.0448	.0056	.0942	.1233	.0001	-.0529	-.0459	.0112	.0951
0.30	.0904	.0022	-.0384	-.0348	.0056	.0729	.0962	-.0009	-.0416	-.0340	.0089	.0705
0.40	.0760	.0014	-.0322	-.0288	.0049	.0601	.0753	-.0006	-.0323	-.0264	.0068	.0545
0.50	.0639	.0011	-.0270	-.0241	.0042	.0534	.0588	-.0006	-.0253	-.0207	.0053	.0424
0.75	.0416	.0008	-.0175	-.0156	.0027	.0326	.0318	-.0002	-.0135	-.0111	.0029	.0230
1.0	.0270	.0005	-.0114	-.0101	.0018	.0212	.0170	-.0002	-.0074	-.0061	.0015	.0124
1.5	.0113	.0002	-.0098	-.0043	.0007	.0089	.0050	.0000	-.0021	-.0018	.0004	.0036
2.0	.0048	.0001	-.0021	-.0018	.0003	.0037	.0014	.0000	-.0006	-.0006	.0001	.0010
3.0	.0008	.0000	-.0004	-.0003	.0000	.0006						

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Floating Block Theory in Structural Analysis*

By STANLEY U. BENSCOTER†

SYNOPSIS

A process of reaction distribution is developed for the purpose of calculating reactions beneath hinged floating blocks. Application to hinged base slabs is illustrated. The analogous correspondence to the process of moment distribution is explained by using the column analogy.

INTRODUCTION

Structures such as retaining walls, gravity dams, piers or column footings are designed on the assumption that the base reaction has a linear distribution. The reaction diagram is considered to be of trapezoidal shape (triangular and rectangular diagrams being special cases). Actually a linear base reaction diagram could be valid only if the structure were floating in a very dense liquid. If the foundation material possesses shearing strength and rigidity, which does not break down and disappear with time, the base reaction must be nonlinear. Justification of this statement may be found both in the theory of elasticity and laboratory and field measurements of base pressures.

Another important requirement for the validity of linear reaction diagrams is that the structure must not have a flexible base slab. A flexible base slab will have a non-linear reaction diagram, even when floating in a liquid, if loaded with any distribution of loading other than uniform. Thus it is seen that floating block theory is now in common usage in the analysis of individual, isolated structures. The present paper presents a method of calculating the base reaction distribution for several floating blocks attached by hinges. The computation method may be called "reaction distribution" since it is performed in the same manner as "moment distribution" for the analysis of continuous beams.

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REACTIONS FOR INDIVIDUAL BLOCKS

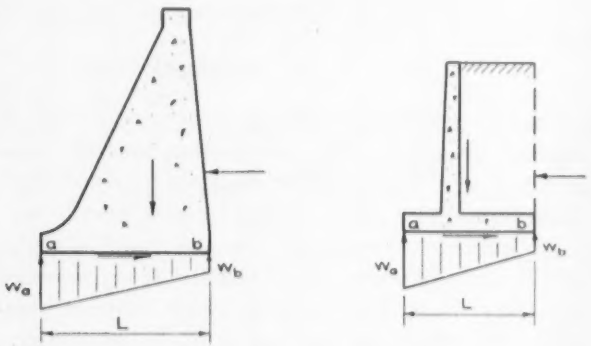
The reactions for a gravity dam or a cantilever retaining wall, as shown in Fig. 1, are readily calculated by considering a slice of the structure one foot thick and using the flexure formula. Considering the origin of coordinates to be at the center of the base, the toe and heel pressures, w_a and w_b, are given by,

$$w_a = \frac{P}{A} + \frac{Mx_a}{I} \quad \text{(1a)}$$

$$w_b = \frac{P}{A} + \frac{Mx_b}{I} \quad \text{(1b)}$$

where P = total vertical load on foundation (positive downwards)
M = moment of P about origin
A = area of base of one foot slice (equals length L)
I = moment of inertia of base area (equals $L^3/12$)
$x_a = -L/2$, $x_b = L/2$

Fig. 1—Linear reaction diagrams



The trapezoidal reaction diagrams would be strictly correct only if the structures were floating in a dense liquid. Thus the flexure formula may be considered as a formula for the reaction distribution of a floating block acted upon by forces and moments. The weight of the floating block may be neglected and the flexure formula will then give the reactions caused by the applied loads.

A more general type of floating block is one having a planform which is not rectangular, such as the one shown in Fig. 2. For the purpose of describing the planform of the floating block and the loading distribution, it is convenient to borrow several words from the terminology which is used by the aeronautical engineer in describing the planform and loading distribution of an airplane wing. The width of the planform is called the chord and the length is called the span. The chord c may be variable as shown in Fig. 2. The distribution of loading across

the planform may be described as the chordwise distribution and the distribution along the planform may be described as the spanwise distribution.

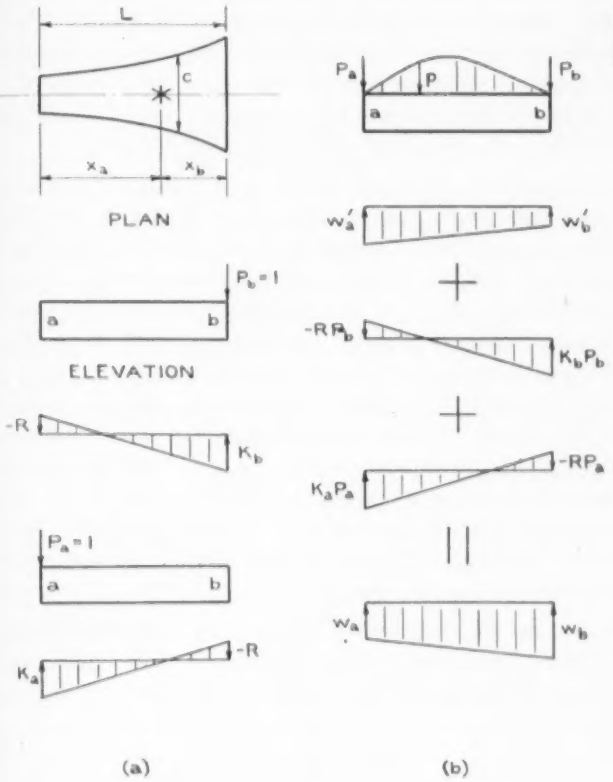


Fig. 2—Floating block with variable chord

Throughout this paper the planform of the floating block is considered to be symmetrical about a center line running in the spanwise direction. When the planform has a variable chord, the area and moment of inertia to be used in the flexure formula for calculating reactions may be computed from the integral formulas,

$$A = \int_0^L c\,dx \quad \text{(2a)}$$

$$I = \int_0^L cx^2\,dx \quad \text{(2b)}$$

where the coordinate x is measured from the centroid of the planform. The chordwise distribution of applied loads and reactions will be considered to be uniform at all sections and such chordwise distribution

diagrams will not be shown. A spanwise distribution diagram and the planform dimensions will serve to define completely the loading magnitude and distribution. The reader who is familiar with the column analogy, as developed by Hardy Cross*, will recognize the similarity in physical concepts between the analogous column and the floating block. Certain fundamental physical properties of an isolated block must be developed for use in the reaction distribution process, just as similar properties of an isolated span of a continuous beam are developed with the aid of the column analogy for use in the moment distribution procedure.

In Fig. 2(a) a floating block is shown with a unit load at the right end. A concentrated load will always be considered as acting on the center line of the planform so that the chordwise distribution of the reaction will always be uniform. The right end reaction ordinate is K_b and the left end reaction ordinate is $-R$. The reaction diagram may be regarded as an influence line for reaction at point b at the right end of the block. Consequently the quantities K_b and R may be called influence numbers. They may be converted to true deflection influence numbers by dividing by the density of the liquid. However, the actual density of the liquid does not enter into any of the analyses of this paper. In Fig. 2(a) there is also shown a unit load acting at point a at the left end of the block. The left end reaction ordinate is indicated as K_a and the right end reaction ordinate is $-R$. The value R arises in both cases in agreement with Maxwell's law of reciprocal deflections.

The quantities K_a and K_b may be called primary influence numbers and R may be called the secondary influence number. The values of the influence numbers may be computed by using Eqs. (1).

$$K_a = \frac{1}{A} + \frac{x_a^2}{I} \quad \text{(3a)}$$

$$K_b = \frac{1}{A} + \frac{x_b^2}{I} \quad \text{(3b)}$$

$$-R = \frac{1}{A} + \frac{x_a x_b}{I} \quad \text{(3c)}$$

In these equations I is the centroidal moment of inertia and x_a and x_b are the distances of points a and b from the centroid of the planform. It may prove to be convenient to express the influence numbers in terms of the planform area thus,

$$K_a = \frac{\alpha_a}{A}, K_b = \frac{\alpha_b}{A}, R = \frac{\beta}{A} \quad \text{(4)}$$

The values of α_a, α_b and β can be readily expressed by introducing the radius of gyration ρ of the planform.

*"The Column Analogy," by Hardy Cross, Bull. No. 215, Eng. Expt. Sta., 1932, Univ. of Ill.

$$\alpha_a = 1 + \left(\frac{x_a}{\rho}\right)^2 \quad \text{(5a)}$$

$$\alpha_b = 1 + \left(\frac{x_b}{\rho}\right)^2 \quad \text{(5b)}$$

$$-\beta = 1 + \frac{x_a x_b}{\rho^2} \quad \text{(5c)}$$

In Fig. 2(b) the same block is shown carrying a continuously distributed load as well as concentrated loads P_a and P_b at either end. Only the spanwise distribution of the loading is shown since the chordwise distribution is uniform at all points along the span. The reaction is shown as consisting of three parts which may be added, or superposed. The first part is a trapezoidal distribution, having end ordinates w'_a and w'_b, and is due to the continuously distributed load. The ordinates w'_a and w'_b are computed from the flexure formula as given by Eq. (1). If p is the applied load per unit of area, the magnitude of the distributed load and its moment about the center of gravity of the planform, are given by,

$$P = \int_o^L pc\,dx \quad \text{(6a)}$$

$$M = \int_o^L pcx\,dx \quad \text{(6b)}$$

The second and third parts of the reaction are due to the loads P_a and P_b. They are proportional to the reaction influence diagrams of Fig. 2(a). Since the chordwise distribution of the reactions is uniform, only the spanwise distribution is shown. When the three parts of the reaction are added, the resultant reaction has a trapezoidal spanwise distribution with end reaction ordinates given by,

$$w_a = w'_a + K_a P_a - R P_b \quad \text{(7a)}$$

$$w_b = w'_b - R P_a + K_a P_b \quad \text{(7b)}$$

These equations may be called the reaction-shear equations and may be recognized as having direct analogous correspondence to the slope-deflection equations for a single span of a continuous beam.

VALIDITY OF THE FLEXURE FORMULA

The flexure formula, as used for the stress analysis of beam-columns, has been given as Eqs. (1) for calculating the reaction distribution. Although the formula is known to be valid for this purpose, a simple proof is of interest. Consider a block with a distributed load p and a linearly distributed reaction w, both in lb. per sq. ft. The net load acting on the block is the difference $(p\text{-}w)$. The load and reaction together must

satisfy the equation of equilibrium of vertical forces and the equation of equilibrium of moments about any point in the plane of symmetry of the block, say the center of gravity of the planform. These two conditions give the following equations,

$$\int_0^L (p\text{-}w)c\,dx = 0 \quad \text{(8a)}$$

$$\int_0^L (p\text{-}w)cx\,dx = 0 \quad \text{(8b)}$$

These integral equations of equilibrium have direct analogous correspondence to the integral equations of continuity which govern the bending moments in a fixed-ended beam. By separating the integrals into two parts and transposing, Eqs. (8) become,

$$\int_0^L wc\,dx = \int_0^L pc\,dx = P \quad \text{(9a)}$$

$$\int_0^L wcx\,dx = \int_0^L pcx\,dx = M \quad \text{(9b)}$$

The formulas of Eqs. (6) have been substituted into Eqs. (9). Since the structural deformation of the floating block is regarded as being negligible, it is known that the reaction w has a linear distribution and may be assumed in the form,

$$w = a + bx \quad \text{(10)}$$

This formula for w may be substituted into Eqs. (9) to obtain two linear algebraic equations which may be solved for a and b. The reader should carry out this suggested procedure*. Another approach is to assume values for a and b, substitute in Eqs. (9), and thus show the assumption to be correct. This procedure will be illustrated. Assume w to be given by the flexure formula according to Eq. (1).

$$w = \frac{P}{A} + \left(\frac{M}{I}\right) x \quad \text{(11)}$$

This assumption is seen to be a linear function of x in agreement with Eq. (10). Substituting rhis into Eqs. (9) gives,

$$\int_0^L \left[\frac{P}{A} + \left(\frac{M}{I}\right) x\right] c\,dx = P \quad \text{(12a)}$$

$$\int_0^L \left[\frac{P}{A} + \left(\frac{M}{I}\right) x\right] cx\,dx = \mathrm{M} \quad \text{(12b)}$$

*The procedure is illustrated for the more general case of unsymmetrical bending in "The Column Analogy," p. 10.

Separating the integrals into two parts and taking the constants outside of the integral signs gives,

$$\frac{P}{A}\int_0^L c\,dx + \frac{M}{I}\int_0^L cx\,dx = \mathrm{P} \quad \ldots\ldots (13a)$$

$$\frac{P}{A}\int_0^L cx\,dx + \frac{M}{I}\int_0^L cx^2 dx = \mathrm{M} \quad \ldots\ldots (13b)$$

Since the origin of coordinates is the center of gravity of the planform, one of the integrals in the equations vanishes.

$$\int_0^L cx\,dx = 0 \quad \ldots\ldots (14)$$

Dropping out the terms containing this integral and substituting the formulas of Eqs. (2), the Eqs. (13) reduce to identities. Since the solution is known to be linear, and is also known to be unique, the assumed solution must be the correct solution. A proof, similar to the above, serves to demonstrate the validity of the column analogy in a simple and convenient manner.

ELEMENTARY EXAMPLES

In Fig. 3 there is shown a number of different loads and reactions acting on a floating block with a rectangular planform. The distributed loadings have a maximum ordinate of p_m lb. per sq. ft. The end ordinates of the reaction diagrams may be expressed in terms of p_m. D. B. Steinman has recently pointed out* a convenient way of remembering the end reaction ordinates for the case of triangular loading. These end ordinates may be computed as the reactions on a simple beam of span L which is loaded with a concentrated load of magnitude p_m acting at the same point as the location of p_m on the floating block.

Several examples are shown in Fig. 3 of reactions due to a concentrated load P acting at one end or both ends of the block. In the case of a concentrated load the area A of the planform enters into the reaction calculations. One of the examples shows P at the right end while the left end of the block is attached by a hinge to a rigid wall. This wall may be thought of as the sidewall of a reservoir containing the liquid in which the block is floating. The hinge prevents the block from sinking at the left end and, hence, the reaction ordinate must be zero at that end. For this case the right end reaction ordinate is found from statics to be $\frac{3}{A}P$.

*"Graphic Methods for Engineers," by D. B. Steinman, *Eng. News-Rec.* V. 132, No. 18, May 4, 1944, pp. 659-661.

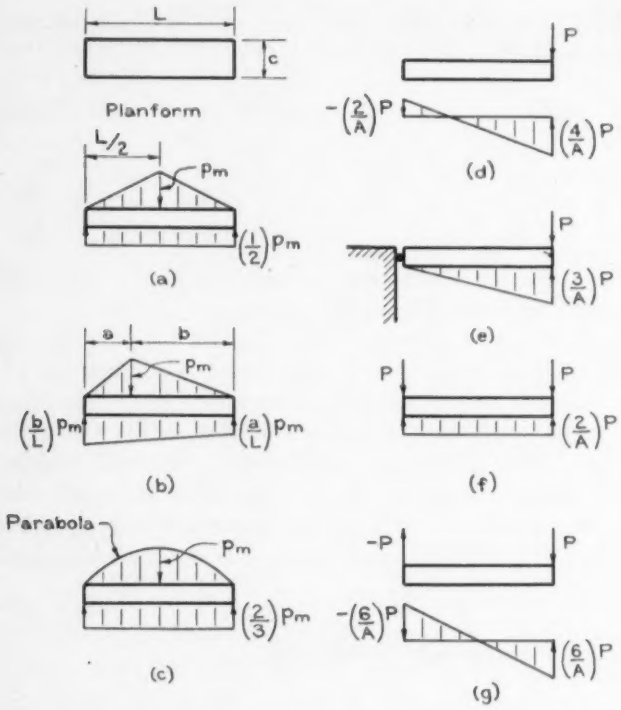


Fig. 3—Floating block with constant chord

The coefficient $3/A$ may be called a modified influence number. It is three-fourths of the standard influence number for a block with a rectangular planform.

In Fig. 4 two blocks are shown with variable chords. The block of Fig. 4(a) has a planform with a chord which varies linearly. The calculated location of the center of gravity is shown. The block is acted upon by a unit load at the right end and, consequently, the reaction ordinates are influence numbers. The area and section moduli are shown with the calculations for the reactions. The block of Fig. 4(b) has a parabolic variation of the chord. It is symmetrical about the center of the span and, hence, the center of gravity is at mid-span. Due to this symmetry the two section moduli are of equal magnitude but of opposite signs.

BLOCKS CONNECTED BY HINGES

The next step in the development of floating block theory is to consider two blocks joined together by a hinge as shown in Fig. 5. Both blocks have the same constant chord but are of unequal spans. The left block has no load but the right block has a load with the spanwise dis-

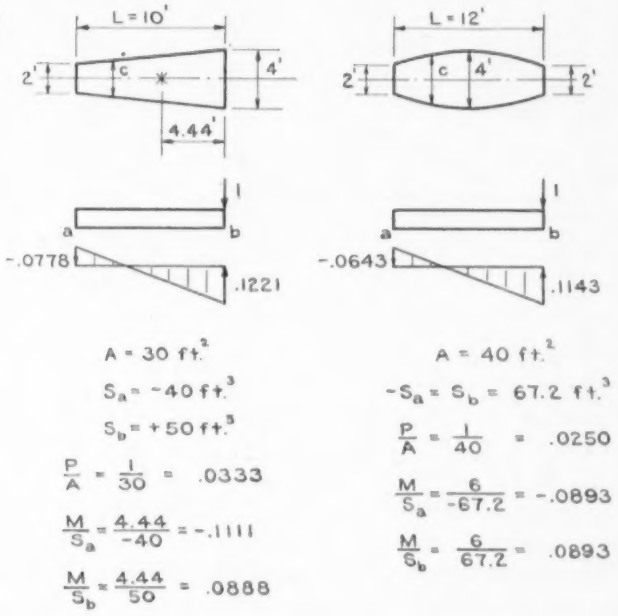


Fig. 4—Calculation of influence numbers

tribution as shown. Consider, temporarily, that the pin of the hinge has been removed so that each block is free to move alone into an equilibrium position. The right block will be displaced downward by the load and will develop a trapezoidal reaction as shown. The left end ordinate w'_a is seen to be two-thirds of p_m and the right end ordinate is one-third of p_m. The values of 10 and 5 lb. per sq. ft. may be called "free end reactions" in analogous correspondence to the "fixed end moments" which enter into a moment distribution analysis.

Consider, now, that a concentrated force P is applied downward on the hinged end of the left block and a force P of equal magnitude is applied upward on the hinged end of the right block. If the magnitude of these artificial forces is gradually increased, the hinge will become realigned at some particular value of P and the pin of the hinge can be replaced. The artificial forces may then be removed and a shear of magnitude P will be transferred through the hinge. In the final equilibrium position of the hinged blocks the left end and right end shears will be,

Left Block $\begin{cases} P_a = P_1 = 0 & \text{(15a)} \\ P_b = P_2 = P & \text{(15b)} \end{cases}$

Right Block $\begin{cases} P_a = -P_2 = -P & \text{(15c)} \\ P_b = P_3 = 0 & \text{(15d)} \end{cases}$

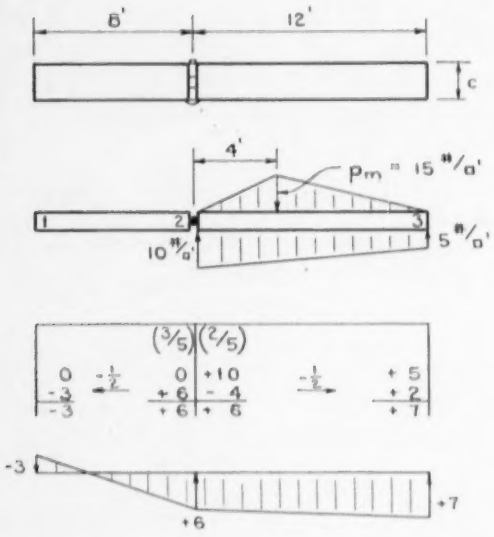


Fig 5—Two hinged blocks

The reaction ordinates for the left block in the final position may be indicated as w_{12} and w_{21}, while the reaction ordinates for the right block will be w_{23} and w_{32}. The use of double subscripts becomes necessary in dealing with more than one block. The influence numbers may also be indicated with double numerical subscripts. The reaction-shear equations for the blocks become (see Eqs. 7),

$$\text{Left Block} \begin{cases} w_{12} = -R_{12}P_2 & \text{(16a)} \\ w_{21} = K_{21}P_2 & \text{(16b)} \end{cases}$$

$$\text{Right Block} \begin{cases} w_{23} = w_{23}' - K_{23}P_2 & \text{(16c)} \\ w_{32} = w_{32}' + R_{32}P_2 & \text{(16d)} \end{cases}$$

The quantities w_{23}' and w_{32}' are the free end reactions. They are the values of w_{23} and w_{32} when $P_2 = P_3 = O$.

In the final position of the blocks, each block will have the same displacement at the hinge. Hence, the reactions w_{21} and w_{23} must be equal. Equating the right sides of Eqs. (16b) and (16c) gives,

$$K_{21}P_2 = w_{23}' - K_{23}P_2 \quad \text{(17a)}$$

$$P_2 = \frac{w_{23}'}{K_{21} + K_{23}} \quad \text{(17b)}$$

Substituting into Eqs. (16) gives,

$$w_{12} = -\left(\frac{R_{12}}{K_{21} + K_{23}}\right) w_{23}' \quad \text{(18a)}$$

$$w_{21} = \left(\frac{K_{21}}{K_{21} + K_{23}}\right) w_{23}' \quad \text{(18b)}$$

$$w_{23} = w_{23}' - \left(\frac{K_{23}}{K_{21} + K_{23}}\right) w_{23}' \quad \text{(18c)}$$

$$w_{32} = w_{32}' + \left(\frac{R_{32}}{K_{21} + K_{23}}\right) w_{23}' \quad \text{(18d)}$$

The following fractions may be defined as distribution factors,

$$d_{21} = \frac{K_{21}}{K_{21} + K_{23}} \quad , \quad d_{23} = \frac{K_{23}}{K_{21} + K_{23}} \quad \text{(19)}$$

The first subscript of the distribution factor indicates the joint and the second subscript indicates the block. The distribution factors are proportional to the influence numbers of the blocks adjacent to the hinge. It is only necessary to know the relative values of the influence numbers K_{21} and K_{23} in order to compute d_{21} and d_{23}. If both blocks have rectangular planforms with the same chord c, the numerical value of c need not be known. If the values of c are unequal, it is only necessary to know their relative values. With equal chords, as in Fig. 5, the distribution factors are inversely proportional to the lengths of the spans.

The following fractions may be defined as carry-over factors,

$$r_{21} = \frac{R_{21}}{K_{21}} = \frac{R_{12}}{K_{21}} \quad , \quad r_{23} = \frac{R_{23}}{K_{23}} = \frac{R_{32}}{K_{23}} \quad \text{(20)}$$

The carry-over factors are calculated as the ratio of the secondary influence numbers to the primary influence numbers. Thus it is only necessary to know relative values of the influence numbers. In the case of rectangular planforms, all carry-over factors have the value ½. The distribution and carry-over factors, as defined, have direct analogous correspondence to the same factors in moment distribution.

Eqs. 18) become,

$$w_{12} = -r_{21} d_{21} w_{23}' = -r_{21} w_{21} \quad \text{(21a)}$$

$$w_{21} = d_{21} w_{23}' \quad \text{(21b)}$$

$$w_{23} = w_{23}' - d_{23} w_{23}' \quad \text{(21c)}$$

$$w_{32} = w_{32}' + r_{23} d_{23} w_{23}' \quad \text{(21d)}$$

From these equations it is seen that the complete spanwise distribution of the reaction can be determined from a knowledge of the free end reactions. The chord of the planform does not need to be known. The shear P_2, which is transmitted through the hinge, is not determined. In order to calculate P_2, it is first necessary to know the numerical value of the chord. From the calculated reactions the force P_2 can then be computed from statics.

The reaction distribution calculations are shown in Fig. 5. The distribution factors for the hinged joint are shown in parentheses. The free end reactions are written first at each end of each block. The unbalanced reaction at the center joint is distributed to the blocks in accord with the values of the distribution factors. The changes in reaction thus brought about at the center joint are multiplied by the carry-over factors, with a negative sign prefixed, to give the change in reaction at the outer ends of the blocks. Addition at each joint gives the three reaction ordinates required to determine the reaction distribution. These computations will readily be seen to be in agreement with Eqs. (21). The negative value of reaction at the left end of the left block may be regarded as being physically possible if it is assumed that the block has sufficient dead weight that it does not lift out of the liquid.

An analysis of three hinged blocks is shown in Fig. 6. The blocks are assumed to have rectangular planforms with equal chords and spans. Consequently the distribution factors and carry-over factors are all equal to ½. The spanwise loading distributions are shown with numerical values of the maximum ordinates. The free end reactions are also shown.

The numerical computations consist of a series of cycles which must be repeated until the desired accuracy is attained. In the example of Fig. 5 the complete solution was obtained from one cycle of computations. A cycle consists of two steps. In the first step the joints are balanced and in the second step the carry-over is performed in all spans. In the example of Fig. 6, two and one-half cycles have been performed to obtain two significant figure accuracy. As each joint is balanced in the first step of a cycle, a horizontal line is drawn to indicate that the joint has been balanced. At the end of the first cycle the hinged joints are out of balance and hence additional cycles are required. The final reaction diagram is illustrated.

HINGED BLOCKS WITH NON-RECTANGULAR PLANFORMS

Fig. 7 illustrates a solution for reactions beneath three hinged blocks with non-rectangular planforms. The blocks have a common center line about which they are all symmetrical. The outer blocks have a chord with linear variation while that of the center block has parabolic variation. The first two blocks have a spanwise load distribution as shown in Fig. 7(a). The first block has a parabolic loading while the second block has a triangular loading. The maximum ordinate of the loading diagrams is at mid-span in both cases.

By using calculus, in accord with Eqs. (9), the magnitude and line of action of each block load may be computed to be as shown in Fig. 7(b).

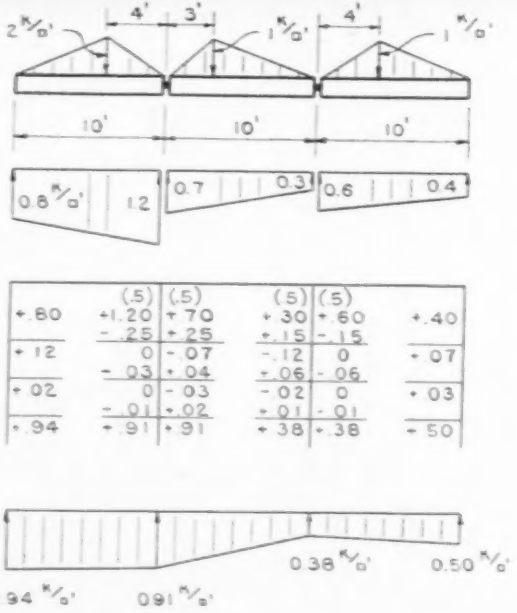


Fig. 6 (above)—Three hinged blocks

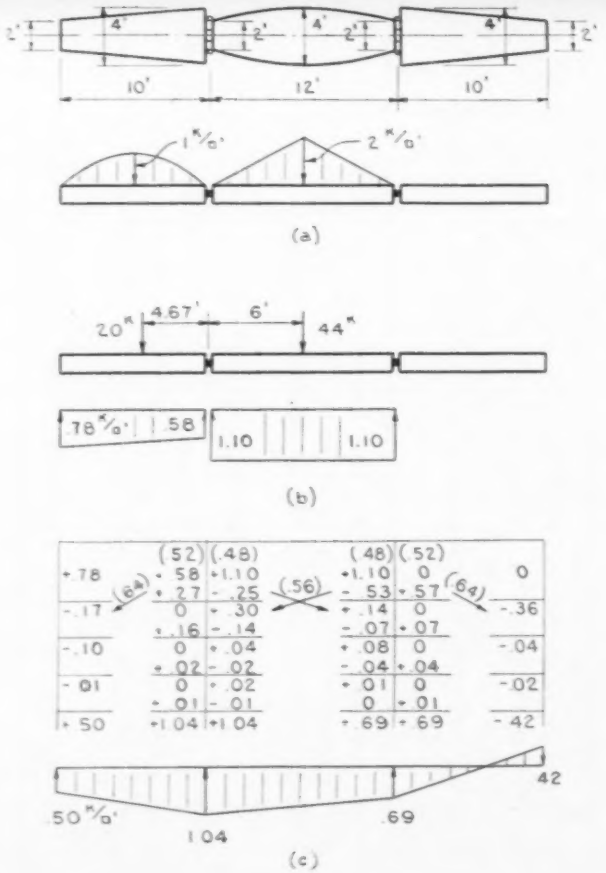


Fig. 7 (right)—Hinged blocks with variable chords

This type of problem could well be used in sophmore calculus courses for engineers to illustrate the application of calculus. It requires the integration of polynomials. The numerical values of the areas and section moduli of these planforms have been given in Fig. 4. From the flexure formula the free end reactions may be computed to be as shown in Fig. 7(b).

The influence numbers which are needed have been computed in Fig. 4. From these influence numbers the distribution and carry-over factors may be computed by using Eqs. (19) and (20). The reaction distribution is shown in Fig. 7(c) with a diagram of the final reaction.

PRACTICAL APPLICATION

An obvious application of reaction distribution is the analysis of a floating bridge which consists of units joined together by hinges. This type of bridge has been used occasionally for both war time and peace time services.

The application which will be illustrated is of more common occurrence. The method of analysis will be applied to concrete structures with hinged base slabs resting on soil. A cross section of a single channel sewage digestion tank is shown in Fig. 8. It is assumed that any reinforcement which crosses the base slab joints is placed at the center of the slab depth and serves only to tie the structure together at these points. Many such structures were built during the war program with no base slab reinforcement at all.

The dimensions of the structure are shown and the magnitudes of the forces have been calculated assuming the concrete to weigh 150 lb per cu. ft. The reaction is first computed for the dead load weight of the concrete alone. A slice of the structure one foot thick is considered in the analysis so that the floating blocks have rectangular planforms with a unit chord.

The eccentricity of the resultant force on either of the outer blocks may be computed to be 0.722 ft. From the flexure formula the free end reactions may be computed to be as shown in Fig. 8(b). Advantage may be taken of the symmetry of the structure about its center line. If a reduced influence number for the center block is used in calculating the distribution factors, no carry-over will be required in the center span and the solution will be obtained from one cycle of distribution computations performed in one end span. The standard influence numbers for the spans are inversely proportional to their lengths. The reduced influence number for the center span is one-half of the standard number as illustrated in Fig. 3(f). The distribution calculations are shown in Fig. 7(c) with the final dead load reaction diagram.

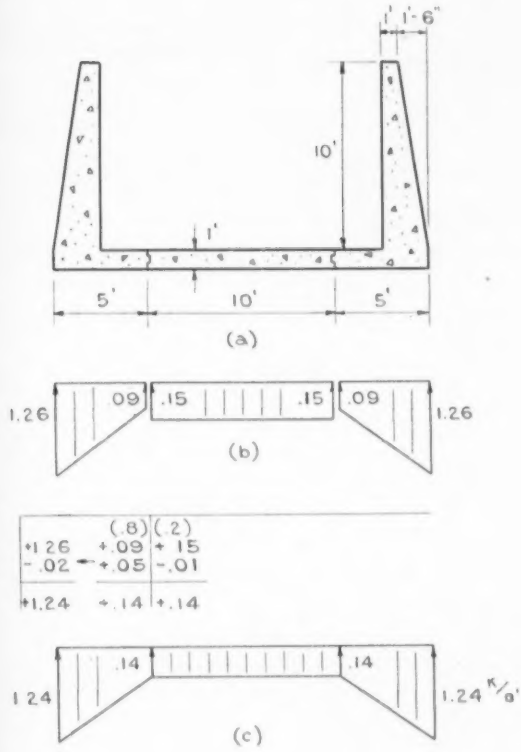


Fig. 8—Sewage tank—analysis for dead load

In Fig. 9 the reaction is calculated for the effect of earth loading only. The soil is assumed to weigh 100 lb. per cu. ft. A triangular wedge of soil on the back of each sidewall weighs 0.75 kips and this force has an eccentricity of 2 ft. The horizontal pressure is assumed to have a "rest" value equal to that of an equivalent liquid weighing 50 lb. per cu. ft. The horizontal thrust in the base slab is assumed to act at mid-depth, giving a moment arm of the horizontal earth pressure of 3.83 ft. The outer floating blocks are acted upon by a thrust and moment from the weight of the soil on the back of the walls and by a pure moment due to the horizontal forces. From the flexure formula the free end reactions due to these loads may be computed to be as shown in Fig. 9(b). The distribution calculations and reaction diagram are shown in Fig. 9(c). The reaction due to earth loads may be added to the dead load reaction to obtain the reaction diagram shown in Fig. 9(d). Internal water loading may be treated in a similar manner. The weight of the water will exert thrusts on each of the blocks and the horizontal water pressure on the walls will exert moments on the outer blocks.

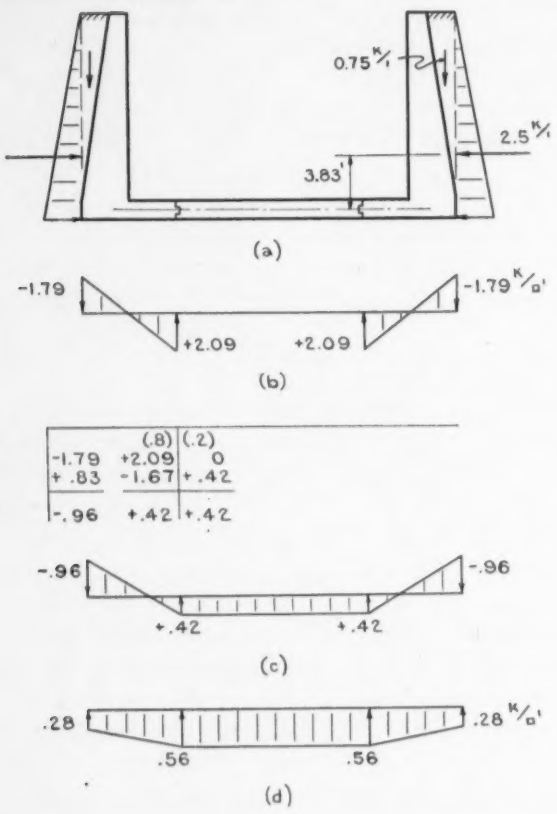
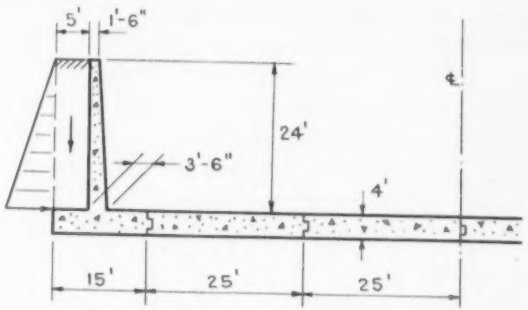


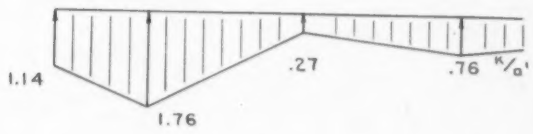
Fig. 9—Sewage tank—analysis for earth loads

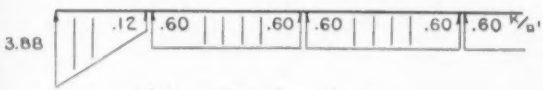
In Fig. 10 is shown a section of a spillway channel which is symmetrical about its center line. The channel floor is constructed of four 25 foot slabs joined by keyed construction joints which act as hinges. The structure rests on soil and does not have reinforcement to carry bending moment across the joints. Concrete and soil weights are assumed as before. The total reaction due to both horizontal and vertical loads is also shown in Fig. 10.

The total reaction has been determined by calculating the effects of the vertical and horizontal loads separately. The reaction is first calculated in Fig. 11 for the dead weight of the concrete and the weight of the soil fill on the heel of the sidewall. The total load on the first block is 30 kips with an eccentricity of 2.36 ft. The free end reactions are shown in Fig. 11(a) as given by the flexure formula. The distribution computations and final reactions are shown in Fig. 11(b) and (c). All influence numbers are inversely proportional to the span lengths and the carry-over factors are all one-half. It is only necessary to perform the dis-

Fig. 10—Spillway channel—loads and reaction





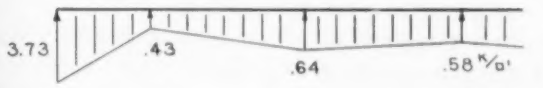


(a) Free End Reactions

Fig. 11—Analysis for vertical loads

	(5/8)	(3/8)	(1/2)	(1/2)	
+3.88	+.12	+.60	+.60	+.60	+.60
0	+.30	-.18	0	0	0
-.15	0	0	+.09	0	0
0	0	0	-.05	+.04	0
0	0	+.02	0	0	-.02
0	+.01	-.01	0	0	0
+3.73	+.43	+.43	+.64	+.64	+.58

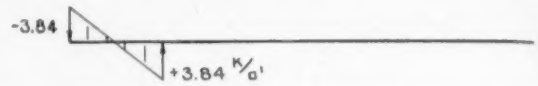
(b) Reaction Distribution



(c) Reaction

tribution computations on half of the structure since the center joint will always be balanced and will not transmit any shear.

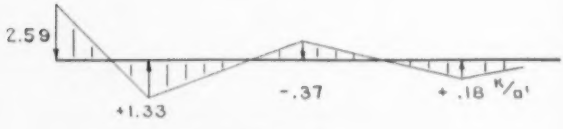
Fig. 12—Analysis for horizontal loads



(a) Free End Reactions

	(5/8)	(3/8)	(1/2)	(1/2)	
-3.84	+3.84	0	0	0	0
0	-2.40	+1.44	0	0	0
+1.20	0	0	-.72	0	0
0	0	0	+.36	-.36	0
0	0	-.18	0	0	+.18
0	-.11	+.07	0	0	0
+.05	0	0	-.03	0	0
0	0	0	+.02	-.01	0
-2.59	+1.33	+1.33	-.37	-.37	+.18

(b) Reaction Distribution



(c) Reaction

The horizontal earth pressure on the sidewalls is again assumed to have a value equal to that of a liquid weighing 50 lb per cu. ft. Acting over a height of 24 ft. the total force is 14.4 kips per ft. The moment arm of the horizontal force about the center of the floor slab is 10 ft., giving a moment on the end block of 144 ft. kips. From the flexure formula the free end reactions are ± 3.84 kips per sq. ft. The reaction distribution for the effect of the horizontal earth pressure is shown in Fig. 12. The total reaction diagram due to vertical and horizontal loads is obtained by adding the reaction diagrams of Fig. 11 and 12.

Considering the trapezoid of reaction beneath the sidewall itself, the resultant force represented by the trapezoid may be calculated to be 21.75 kips, or 73% of the weight of the sidewall and fill on its heel. This means that 27% of the weight of the sidewall and fill is transmitted through the first hinged joint. The joint key must be designed to transmit this shear. It has been common, but wasteful, practice to neglect this shear force in the past.

CONTINUOUS BEAM ANALYSIS

In developing the process of reaction distribution a number of instances have been specifically noted in which there is a direct analogous

correspondence with the analysis of continuous beams. In order to analyze a continuous beam it is first necessary to consider each span individually. The primary and secondary stiffness values must be computed. If the method of moment distribution is to be employed, the fixed end moments must be computed. These individual span quantities may readily be computed by using the concepts of the column analogy. For each span there is an analogous column which may also be regarded as a floating block. The chord of the planform of the analogous column, or floating block, is equal to the value of 1/EI for the beam. The spanwise distribution of loading on the blocks corresponds to the simple beam bending moment diagrams for the beam spans. If these blocks are joined by hinges, the calculation of reactions corresponds to the process of moment distribution. The correspondence between hinged block, or analogous column, analysis and continuous beam analysis was noted by Prof. Hardy Cross*. The various elements of correspondence are as follows:

(1)	Reaction distribution	Moment distribution
(2)	Floating Block	Analogous column
(3)	Span	Span
(4)	Cord c	1/EI
(5)	Spanwise load distribution	Simple beam moment diagram
(6)	Free end reactions	Fixed end moments
(7)	Hinge shear	Joint rotation
(8)	Influence numbers	Stiffness values
(9)	Distribution factors	Distribution factors
(10)	Carry-over factors	Carry-over factors
(11)	Reaction-shear equations	Slope-deflection equations
(12)	Joint reaction ordinate	Joint moment
(13)	Hinge	Simple support
(14)	Free end	Fixed end
(15)	Integral equations of equilibrium	Integral equations of continuity

By substituting the above correspondence the development of reaction distribution as given in this paper becomes a development of moment distribution and the column analogy as applied to beams. The development of hinged block analysis requires only the concept of equilibrium of forces while the development of continuous beam anlysis requires, in addition, the more difficult concept of continuity. Consqeuently it may be possible that the structural engineering student could profitably be introduced to reaction distribution as a prelude to the study of continuous beam analysis.

A few examples of continous beams and the analogous floating blocks will be shown. In Fig. 13 a continuous beam of constant section over three spans is shown. The analogous hinged blocks are also shown with a planform cord of $1/EI$. The spans and loads are so chosen that the simple beam moment diagrams will be the same as the spanwise distribution diagrams of the example of Fig. 6. Consequently the reaction

*"The Column Analogy," p. 55.

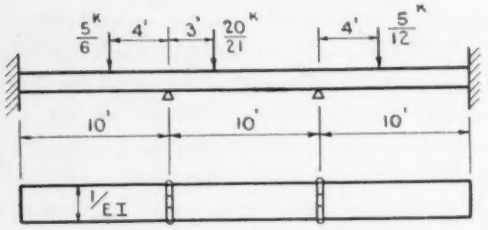


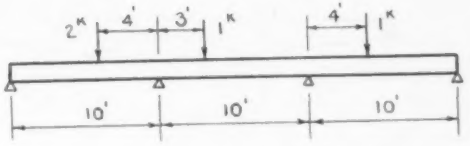
Fig. 13—Continuous beam and analogous floating block

distribution calculations of Fig. 6 may be regarded as moment distribution for the beam of Fig. 13.

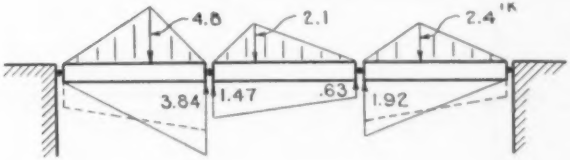
In Fig. 14(a) is shown a prismatic three span beam with four simple supports. Thus there are four hinges on the three floating blocks as shown in Fig. 14(b). The outer hinges are attached to fixed sidewalls of the channel of liquid. The blocks are loaded with simple beam moment diagrams. The free end reaction for the center block is shown by solid lines and for the end blocks is indicated by dotted lines. All four pins must be removed from the four hinges to allow the blocks to move downwards and develop the free end reactions. It is convenient, before the reaction distribution is performed, to lift the outer ends of the outer blocks until the outer hinges become realigned and insert these two pins so that a shear is transferred into the sidewalls. This reduces the reaction ordinates at these two points to zero. The change from free end value to zero causes a change of half as much as the other end of the block, since the carry-over factor is one-half. The calculations for determining the maximum ordinate of the triangular reaction diagrams of the outer blocks are shown in Fig. 14(b). The computations of Fig. 14(c) may be regarded either as moment distribution for the continuous beam or reaction distribution for the floating blocks. A modified influence number is used for the end spans in computing the distribution factors. This influence number is three-fourths of the standard value as shown in Fig. 3(e). The calculated floating block reaction diagram is also shown in Fig. 14(c). This diagram gives the negative moment diagram to be superimposed on the simple beam moment diagrams to obtain the solution of the continuous beam.

An example of a non-prismatic beam is shown in Fig. 15 with the associated floating blocks. The chord of the planform of each block is proportional to $1/EI$ of the beam at corresponding points. On the assumption that cross sections of the beam are rectangular, it is only necessary to know the relative depths of the beam at various points. The bending moments can be completely calculated without knowing the actual beam dimensions if the relative values are known.

Fig. 14—Prismatic beam on simple supports



(a)



+1.92	+2.88	+1.47	+0.63	+1.44	+0.96
-1.92 →	.96			+ .48 ←	-0.96
0	+3.84	+1.47	+0.63	+1.92	0

(b)

(3/7)	(4/7)	(4/7)	(3/7)
+3.84	+1.47	+ .63	+1.92
- 1.02	+1.35	+ .74	- .55
0	- .37	- .68	0
- .16	+ .21	+ .39	- .29
0	- .19	- .10	0
- .08	+ .11	+ .06	- .04
0	- .03	- .05	0
- .01	+ .02	+ .03	- .02
+2.57	+2.57	+1.02	+1.02

(c)

SIGN CONVENTION

In order to study the algebra of hinged floating blocks it is desirable to use a different sign convention for the hinge shear than that which was used in wiriting the reaction-shear equations. It is convenient in dealing with hinged blocks to define the end shears, or hinge shears, to be positive when they act as shown in Fig. 16 for joint 3 and block number 34. The reaction-shear equations for block 34 will appear as,

$$w_{34} = w_{34}' - K_{34}P_3 - R_{34}P_4 \quad \text{(22a)}$$

$$w_{43} = w_{43}' + R_{43}P_3 + K_{43}P_4 \quad \text{(22b)}$$

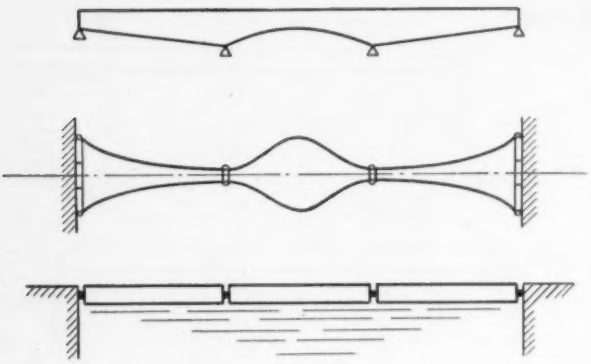
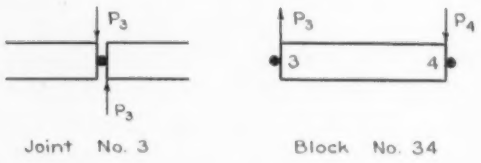


Fig. 15—Non-prismatic girder

Fig. 16—Positive hinge shears



This sign convention, and form of the equations, is in agreement with a bending moment sign convention for continuous beams if the secondary stiffness R (or influence number) is regarded as a positive constant.

CONCLUSION

Using the well known formulas for linear base reaction diagrams as a basis, a method of analysis has been developed for hinged base slabs or hinged floating bridges. The numerical distribution process may be called reaction distribution. Its analogous correspondence to moment distribution and the column analogy has been explained. Since the process involves only the simple concept of equilibrium of forces, it may serve usefully as an introduction to a study of continuous beam analysis by moment distribution.

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Shrinkage and Plastic Flow of Pre-Stressed Concrete*

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Members American Concrete Institute

SYNOPSIS

This paper presents the results of shrinkage and plastic flow measurements on pre-stressed and unstressed specimens for the duration of a year. Stored at 70 F and 50 per cent relative humidity the shrinkage of unstressed specimens reached maximum strain values of 8.7×10^{-4} in. per in. for the concrete and 6.5×10^{-4} for gunite. After the age of 10 days the gunite shrinkage was about 75 per cent of the concrete strains.

The loaded specimens were stressed to approximately 930 psi (low), 1500 psi (intermediate), and 2400 psi (high) for concrete and gunite whose ultimate compressive strengths were 4900 psi and 4500 psi respectively. As shrinkage and plastic flow occurred these stresses decreased until, at the age of 1 year, the stresses in the concrete were 25 to 33 per cent their initial values and the stresses in the gunite were about 43 per cent of the initial. Plastic flow is defined as the difference between the total strain of the loaded specimen and the shrinkage strain of the unstressed specimen during the same time interval. Plastic flow is assumed equal to the stress multiplied by a flow coefficient c. At 380 days the flow coefficients varied from 64×10^{-8} to 79×10^{-8} for the concrete and from 50×10^{-8} to 61×10^{-8} for the gunite. For the low stress specimens the gunite coefficient is 90 per cent of the concrete; for the other two stresses the gunite coefficient is about 75 per cent that of the concrete.

During the period of plastic flow the shrinkage of the unstressed concrete specimens relative to the total deformation of the loaded specimens was 83 percent for low stress, 64 percent for intermediate, and 50 percent for high stress. The corresponding ratios for the gunite were 65 percent for low stress, 59 percent for intermediate, and 53 percent for the high stressed specimens.

INTRODUCTION

A large number of tests have been reported on the deformation of concrete under constant loading conditions but very little information is

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available regarding the behavior of prestressed specimens for which the load is not constant but varies with time and conditions of exposure and use. To evaluate the effect of all of the variables that might enter into consideration in the application of pre-stressed concrete would be a tremendous task. However, it is possible to study the behavior of prestressed specimens under conditions that are probably the worst that could be conceived for field usage and thereby obtain information that would be valuable for use in design.

The purpose of the work reported was twofold, (1) the determination of the rate and probable total amount of stress decrease in the concrete and (2) the contribution made by shrinkage to the decrease in prestress. Accordingly measurements of deformation were made on both stressed and unstressed specimens. Specimens of both concrete and gunite were used.

PROCEDURE IN MAKING TEST SPECIMENS

The gunite was "shot" on the job as a 1:4 mixture using standard equipment for this work. The concrete was mixed in the laboratory with the following quantities of materials per cu. yd., 600 lb. cement, 1260 lb. sand, 1920 lb. of ¾-in. gravel and 36 gal. of water. The compressive strength at 28 days of 4 in. x 8 in. gunite cylinders was 4900 psi and for 6 in. x 12 in. concrete cylinders was 4500 psi.

Gunite specimens were made by "shooting" the material on to 12 in. planks, 16 ft. long, in a thickness of about 5 in. The test bars were then cut out of this column of material, using cutters devised for the purpose. Brass insert bars had been secured to the plank in proper location and position before shooting started and holes were drilled in proper locations on the other three sides of the bars, before the gunite became completely hardened, for the easy placement of other gage point insert bars after the sections were removed to the laboratory. Twenty four hours after the bars were fabricated, they were placed in damp sand and removed to the laboratory where they were fitted with inserts and gage points for reading of deformations.

The concrete specimens were cast in the laboratory in watertight molds, the gage point inserts being cast into the specimen. These specimens were removed from the molds at 24 hours. Both the gunite and concrete specimens were stored in the damp room at 70 F and 95+ per cent relative humidity until removal to the constant temperature room. It was thought desirable to have a period of storage in the constant temperature room before and during loading as the specimens would then have a water content at the time of loading more consistent with that of similar material in the field. After removal to the constant tem-

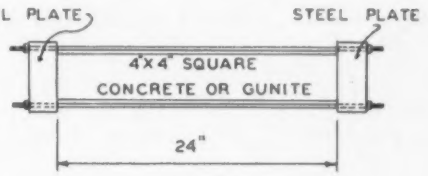

Fig. 1—Method for loading bars

HIGH LOADING

$6-\frac{1}{2}$" rods.

2 plates $6" \times 6" \times 2\frac{1}{2}"$

INTERMEDIATE LOADING

$4-\frac{1}{2}$" rods.

2 plates $6" \times 4\frac{7}{8}" \times 1\frac{3}{4}"$

LOW LOADING

$4-\frac{3}{8}$" rods.

2 plates $5\frac{1}{2}" \times 4\frac{1}{2}" \times 1\frac{1}{4}"$

perature room, maintained at 70 F and 50 per cent relative humidity, the specimens remained there for the balance of the time of reading.

Specimens were loaded to the stresses indicated by means of plates and rods as shown in Fig. 1. A two-day period for application of the load was chosen arbitrarily as being representative of the time for the average prestressing operation in the field, where bars are used. Gage points were provided in the rods used for prestressing, with the same 10-inch gage length used for the concrete and gunite bars. At the end of two days the full prestress was restored by a second tightening of the nuts. This corresponds to the final tightening operation in the field. The rods were stressed by tightening the nuts, care being taken not to twist the rod during tightening. Readings were taken with a Whittemore fulcrum-plate strain gage, values being estimated to hundred thousandths. Readings were taken on four sides of the bar and on all rods for each specimen at variable times, but sufficiently often to have an ample number of readings to fully identify the curves.

Shrinkage readings were taken on unstressed specimens, of the same size, that were subjected to the same storage conditions as the stressed specimens. As with the stressed specimens, readings were made on four sides of the specimen.

RESULTS

Shrinkage of unstressed specimens.

The average shrinkage strains of the unloaded specimens are recorded

TABLE 1—DATA ON SHRINKAGE SPECIMENS

CONCRETE SPECIMENS SIZE 4X4X24"					
DAYS AGE	LOC.	AVE. ΔL	REVISED SHRINKAGE ΔL		
			LOW	INTERD	HIGH
1	LAB.	0.000000	—	—	—
5	M.R.	0.000056	—	—	—
9	CTH	0.000077	0.000000	—	—
12	CTH	0.000177	0.000100	0.00000	—
16	CTH	0.000257	0.000180	0.000080	0.00000
19	CTH	0.000335	0.000257	0.000157	0.000077
27	CTH	0.000467	0.000390	0.000290	0.000210
34	CTH	0.000536	0.000459	0.000359	0.000279
41	CTH	0.000597	0.000520	0.000419	0.000340
48	CTH	0.000637	0.000560	0.000460	0.000380
55	CTH	0.000664	0.000586	0.000486	0.000406
69	CTH	0.000709	0.000632	0.000531	0.000452
97	CTH	0.000764	0.000687	0.000586	0.000507
119	CTH	0.000785	0.000708	0.000607	0.000528
140	CTH	0.000801	0.000723	0.000623	0.000554
200	CTH	0.000817	0.000740	0.000639	0.000560
280	CTH	0.000845	0.000768	0.000667	0.000588
400	CTH	0.000872	0.000795	0.000694	0.000615

GUNITE SPECIMENS SIZE 4X4X24"					
DAYS AGE	LOC	AVE. ΔL	REVISED SHRINKAGE ΔL		
			LOW	INTERD	HIGH
5	LAB.	0.00000	—	—	—
6	M.R.	0.000056	—	—	—
9	C.T.H.	0.000040	0.000000	—	—
12	CTH	0.000129	0.000089	0.00000	—
17	CTH	0.000200	0.000160	0.000071	0.00000
21	CTH	0.000255	0.000215	0.000126	0.000055
28	CTH	0.000336	0.000283	0.000197	0.000126
35	CTH	0.000383	0.000343	0.000254	0.000183
43	CTH	0.000422	0.000382	0.000291	0.000222
50	CTH	0.000462	0.000422	0.000333	0.000262
64	CTH	0.000519	0.000479	0.000390	0.000319
79	CTH	0.000553	0.000513	0.000424	0.000353
92	CTH	0.000572	0.000532	0.000443	0.000372
113	CTH	0.000582	0.000542	0.000453	0.000382
140	CTH	0.000601	0.000561	0.000472	0.000401
200	CTH	0.000623	0.000583	0.000494	0.000423
280	CTH	0.000638	0.000598	0.000509	0.000438
400	CTH	0.000654	0.000614	0.000525	0.000454

NOTE—Revised shrinkage values are determined from the shrinkage average data using reloading dates of stressed specimens of low-intermediate-high loads to determine zero values.

LOC. = Place of reading ΔL is expressed in inches per inch

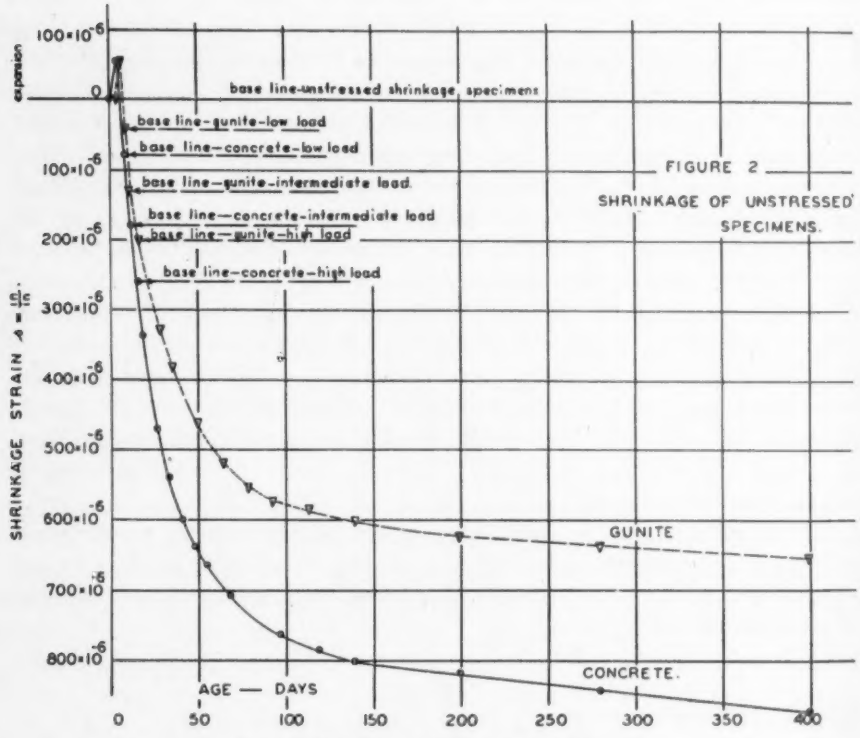

FIGURE 2
SHRINKAGE OF UNSTRESSED SPECIMENS.

Fig. 2—Shrinkage of unstressed specimens

in Table 1 and plotted in Fig. 2. The initial (zero) reading was taken at the age of 1 day for the concrete and at 5 days for the gunite. Both showed an immediate expansion which was rapidly reduced as the specimens dried out in the 50 per cent relative humidity of the storage room. The rate of shrinkage was the same for both types of concrete for 3 or 4 days; thereafter the concrete decreased in length much more rapidly. At the end of 400 days the total shrinkage of the gunite was 75 per cent of that of the concrete.

The shrinkage of the stressed specimens subsequent to loading is assumed to be the same as the unstressed specimens in the same time interval. The bases from which these shrinkages are computed are indicated in Fig. 2.

The maximum shrinkage strains of 8.7×10^{-4} for the concrete and 6.5×10^{-4} for the gunite appear to be reasonable values for the rather high cement content and the relatively low humidity.

Deformation of loaded specimens.

The average strains recorded for about 15 specimens of each load are listed in Table 2 and are plotted in Fig. 3 to 5. The concrete strains record shrinkage to the time of loading, the elastic strain due to application of the load and the subsequent deformations in the two-day interval before the reloading. At "reloading" the full prestress is restored by additional tightening of the nuts. The reloading elastic strain is given and the additional strains up to 400 days while the concrete stress decreased. The steel strain readings were commenced when the bars were tightened to give the initial loading. These strains determined the total force in the steel bars and, hence, the stress in the concrete. The modulus of elasticity of the steel was determined by test to be 29.3×10^6 psi for the $\frac{3}{8}$-in. round bars and 28.4×10^6 psi for the $\frac{1}{2}$-in. round bars.

These loaded specimens experienced only shrinkage strains until the loads were applied. The concrete shrinkages, both expansion and contraction, closely paralleled those recorded for the unstressed specimens. Upon application of the load, or upon reloading, elastic strains took place immediately. The elastic strains have been used to compute the secant Moduli of Elasticity, which are tabulated below. For comparison the table also includes the tangent Modulus of Elasticity obtained by sonic tests of the specimens immediately prior to critical loading. The tangent moduli are, of course, higher values than the secant moduli.

Subsequent to the reloading, Fig. 3 and 4 show the additional strains as time increased and the concrete stresses decreased. These curves are all similar. If the three concrete curves of Fig. 3 are so placed that their terminals at 400 days are coincident, they are identical between the ages of 70 days and 400 days, though the stresses for the three loadings

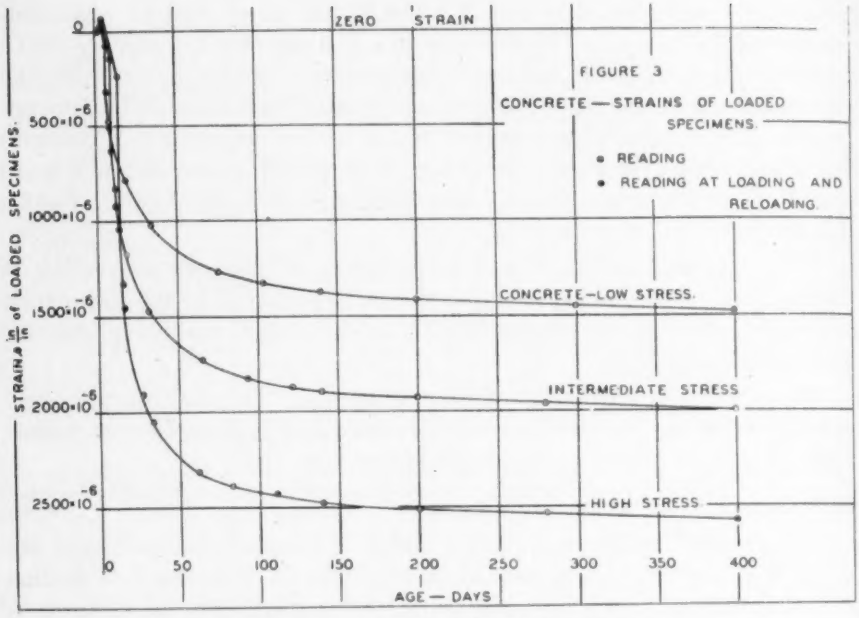

Fig. 3—Concrete—strains of loaded specimens

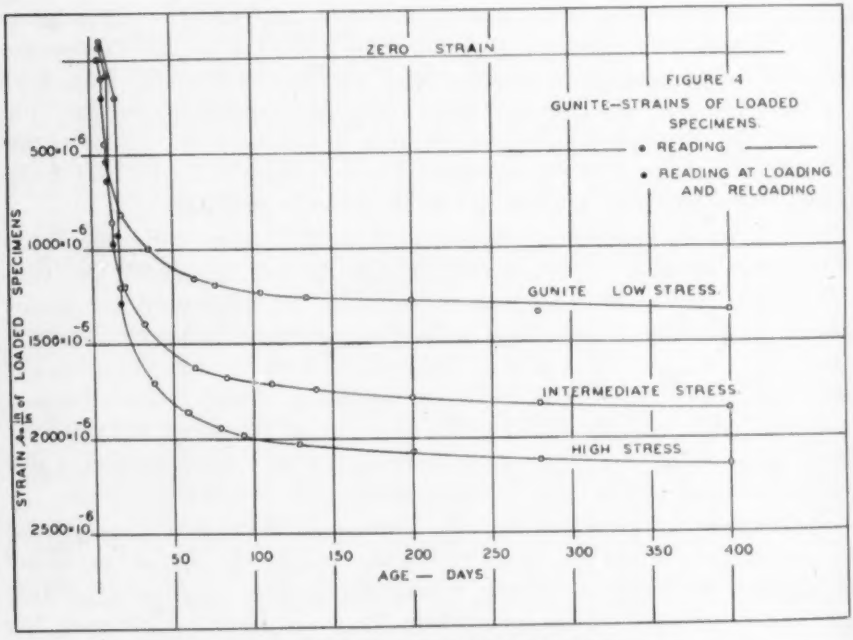

Fig. 4—Gunite—strains of loaded specimens

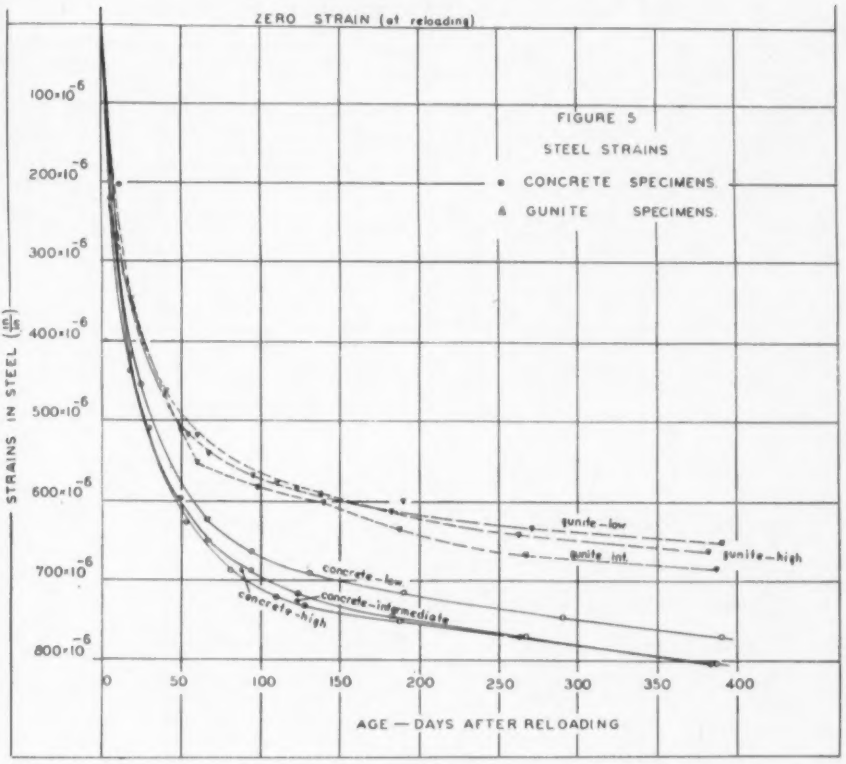

Fig. 5—Steel strains

are quite different in the time interval. During the same time interval, the shrinkage strains of the unstressed specimens was about 70 per cent as much. When the same procedure is applied to the gunite data, (Fig. 4), the intermediate and high strain curves coincide from 90 to 400 days. The low-stressed curve coincides with the other two only between 280 and 400 days. Between 90 and 280 days the low-stress contractions are about two-thirds as much as the others. The shrinkage of the unstressed specimens was about one-half that of the higher-stressed specimens between 90 and 400 days.

Deformations subsequent to removal of load.

Table 3 records the average strains of those specimens whose loads were removed at 90 days, or 200 days, or 400 days. These strains are not identical with those listed in Table 2, because Table 2 records the cumulative average of 15 to 9 specimens (including those of Table 3). Using the stress-strain data for the individual specimens, the secant modulus of

TABLE 2—DATA ON DEFORMATION OF SPECIMENS

CONCRETE — LOW STRESS				
TIME OF READING	DAYS	AVERAGE ΔL		
		CONCRETE	STEEL	STRESS
TO M.R.	1	+ 0	+ 0	0
M.R. to C.T.H.	5	+0.000046	+ 0	0
C.T.H.	7	-0.000075	+ 0	0
C.T.H. LOAD	7	-0.000320	+0.001123	940
C.T.H.	9	-0.000479	+0.000920	770
C.T.H. RELD	9	-0.000514	+0.001118	935
C.T.H.	18	-0.000784	+0.000850	711
C.T.H.	34	-0.001019	+0.000662	555
C.T.H.	76	-0.001267	+0.000494	414
C.T.H.	104	-0.001322	+0.000455	381
C.T.H.	140	-0.001368	+0.000427	358
C.T.H.	200	-0.001409	+0.000403	337
C.T.H.	300	-0.001444	+0.000374	313
C.T.H.	400	-0.001474	+0.000348	291

CONCRETE — INTERMEDIATE STRESS				
TIME OF READING	DAYS	AVERAGE ΔL		
		CONCRETE	STEEL	STRESS
TO M.R.	1	+ 0	+ 0	0
M.R. to C.T.H.	5	+0.000064	+ 0	0
C.T.H.	10	-0.000141	+ 0	0
C.T.H. LOAD	10	-0.000624	+0.001090	1550
C.T.H.	12	-0.000828	+0.000861	1222
C.T.H. RELD	12	-0.000914	+0.001090	1550
C.T.H.	18	-0.001171	+0.000870	1238
C.T.H.	31	-0.001470	+0.000672	954
C.T.H.	66	-0.001731	+0.000492	696
C.T.H.	94	-0.001822	+0.000403	572
C.T.H.	122	-0.001867	+0.000371	526
C.T.H.	140	-0.001889	+0.000367	509
C.T.H.	200	-0.001920	+0.000343	484
C.T.H.	280	-0.001953	+0.000319	454
C.T.H.	400	-0.001992	+0.000289	408

CONCRETE — HIGH STRESS				
TIME OF READING	DAYS	AVERAGE ΔL		
		CONCRETE	STEEL	STRESS
TO M.R.	1	+ 0	+ 0	0
M.R. to C.T.H.	5	+0.000065	+ 0	0
C.T.H.	14	-0.000233	+ 0	0
C.T.H. LOAD	14	-0.001040	+ 0.001120	2390
C.T.H.	16	-0.001323	+ 0.000851	1811
C.T.H. RELD	16	-0.001451	+ 0.001120	2389
C.T.H.	27	-0.001905	+ 0.000766	1955
C.T.H.	34	-0.002046	+ 0.000684	1459
C.T.H.	62	-0.002302	+ 0.000523	1113
C.T.H.	83	-0.002380	+ 0.000470	1004
C.T.H.	111	-0.002433	+ 0.000435	926
C.T.H.	140	-0.002470	+ 0.000409	865
C.T.H.	200	-0.002504	+ 0.000377	803
C.T.H.	280	-0.002533	+ 0.000352	750
C.T.H.	400	-0.002570	+ 0.000319	680

GUNITE — LOW STRESS				
TIME OF READING	DAYS	AVERAGE ΔL		
		CONCRETE	STEEL	STRESS
TO M.R.	5	0	0	0
M.R. to C.T.H.	6	+0.000082	0	0
C.T.H.	7	-0.000096	0	0
C.T.H. LOAD	7	-0.000280	+0.001003	790
C.T.H.	9	-0.000444	+0.000825	651
C.T.H. RELD	9	-0.000525	+0.001162	916
C.T.H.	18	-0.000815	+0.000913	720
C.T.H.	35	-0.001002	+0.000780	598
C.T.H.	64	-0.001157	+0.000646	510
C.T.H.	77	-0.001190	+0.000623	491
C.T.H.	105	-0.001229	+0.000594	469
C.T.H.	133	-0.001256	+0.000578	455
C.T.H.	200	-0.001279	+0.000564	445
C.T.H.	280	-0.001343	+0.000529	418
C.T.H.	400	-0.001335	+0.000511	404

GUNITE — INTERMEDIATE STRESS				
TIME OF READING	DAYS	AVERAGE ΔL		
		CONCRETE	STEEL	STRESS
TO MR	5	0	0	0
MR to C.T.H.	6	+0.000079	0	0
C.T.H.	10	-0.000082	0	0
C.T.H. LOAD	10	-0.000641	+0.001121	1483
C.T.H.	12	-0.000858	+0.000901	1191
C.T.H. RELD	12	-0.000969	+0.001171	1550
C.T.H.	18	-0.001198	+0.000992	1310
C.T.H.	31	-0.001385	+0.000826	1092
C.T.H.	63	-0.001618	+0.000682	875
C.T.H.	83	-0.001672	+0.000620	820
C.T.H.	111	-0.001706	+0.000591	781
C.T.H.	140	-0.001740	+0.000572	756
C.T.H.	200	-0.001786	+0.000538	711
C.T.H.	280	-0.001827	+0.000505	667
C.T.H.	400	-0.001859	+0.000490	648

GUNITE — HIGH STRESS				
TIME OF READING	DAYS	AVERAGE ΔL		
		CONCRETE	STEEL	STRESS
TO M.R.	5	0	0	0
M.R. to C.T.H.	6	+0.000091	0	0
C.T.H.	15	-0.000195	0	0
C.T.H. LOAD	15	-0.000986	+0.001173	2425
C.T.H.	17	-0.001915	+0.000956	1981
C.T.H. RELD	17	-0.001282	+0.001158	2390
C.T.H.	37	-0.001698	+0.000814	1686
C.T.H.	58	-0.001851	+0.000695	1442
C.T.H.	79	-0.001937	+0.000642	1330
C.T.H.	93	-0.001974	+0.000616	1279
C.T.H.	128	-0.002022	+0.000580	1203
C.T.H.	156	-0.002045	+0.000568	1177
C.T.H.	200	-0.002075	+0.000547	1134
C.T.H.	280	-0.002115	+0.000516	1072
C.T.H.	400	-0.002143	+0.000497	1030

NOTE—Reld = Reloaded; ΔL = Deformation of specimen expressed in inches per inch; Days = Age of specimen; Stress = Pounds per square inch. in concrete; M.R. = Moisture room; C.T.H. = Constant temperature-humidity room.

TABLE 3—DEFORMATION CURVE DATA

NOTE: These figures represent the average of three specimens for their respective unloading date. Each ΔL, therefore, is calculated from its respective three specimens.

TIME OF READING	CONCRETE SPECIMENS — LOW STRESS — 935 PSI.						CONCRETE SPECIMENS — INTERMEDIATE STRESS—1550 PSI.					
	90 DAY		200 DAY		400 DAY		90 DAY		200 DAY		400 DAY	
	DYS	ΔL	DYS	ΔL	DYS	ΔL	DYS	ΔL	DYS	ΔL	DYS	ΔL
END OF CURING PERIOD	5	+0.000064	5	+0.0000044	5	+0.0000076	5	+0.0000048	5	+0.0000087	5	+0.0000099
SPECIMENS LOADED	7	−0.000374	7	−0.000281	7	−0.000316	10	−0.000720	10	−0.000564	10	−0.000596
SPECIMENS RELOADED	9	−0.000595	9	−0.000472	9	−0.000503	12	−0.001031	12	−0.000852	12	−0.000844
BEFORE UNLOADING	90	−0.001441	200	−0.001264	400	−0.001509	90	−0.001967	200	−0.001745	400	−0.001880
ELASTIC RECOVERY	90	−0.001347	200	−0.001186	400	−0.001426	90	−0.001795	200	−0.001591	400	−0.001774
ELASTIC AFTER EFFECT	104	−0.001327	214	−0.001169	414	−0.001411	104	−0.001739	214	−0.001564	414	−0.001752

TIME OF READING	CONCRETE SPECIMENS — HIGH STRESS — 2390 PSI.						GUNITE SPECIMENS — LOW STRESS — 935 PSI					
	90 DAY		200 DAY		400 DAY		90 DAY		200 DAY		400 DAY	
	DYS	ΔL	DYS	ΔL	DYS	ΔL	DYS	ΔL	DYS	ΔL	DYS	ΔL
END OF CURING PERIOD	5	+0.0000089	5	+0.0000052	5	+0.0000062	6	+0.0000065	6	+0.0000056	6	+0.0000063
SPECIMENS LOADED	14	−0.0001103	14	−0.0000975	14	−0.0001111	7	−0.0000259	7	−0.0000279	7	−0.0000289
SPECIMENS RELOADED	16	−0.0001558	16	−0.0001292	16	−0.0001550	9	−0.0000507	9	−0.0000518	9	−0.0000527
BEFORE UNLOADING	90	−0.0002547	200	−0.0002302	400	−0.0002719	105	−0.0001188	200	−0.0001264	400	−0.0001358
ELASTIC RECOVERY	90	−0.0002302	200	−0.0002069	400	−0.0002528	90	−0.0001055	200	−0.0001140	400	−0.0001234
ELASTIC AFTER EFFECT	104	−0.0002252	214	−0.0002030	414	−0.0002499	104	−0.0001023	214	−0.0001107	414	−0.0001171

TIME OF READING	GUNITE SPECIMENS. — INTERMEDIATE STRESS—1550 PSI						GUNITE SPECIMENS. — HIGH STRESS — 2390 PSI.					
	90 DAY		200 DAY		400 DAY		90 DAY		200 DAY.		400 DAY.	
	DYS.	ΔL	DYS	ΔL	DYS	ΔL	DYS	ΔL	DYS	ΔL	DYS	ΔL
END OF CURING PERIOD	6	+0.0000055	6	+0.0000085	6	+0.0000080	6	+0.0000088	6	+0.0000082	6	+0.0000094
SPECIMENS LOADED	10	−0.0000598	10	−0.0000631	10	−0.0000599	15	−0.0000981	15	−0.0001076	15	−0.0000983
SPECIMENS RELOADED	12	−0.0000932	12	−0.0000945	12	−0.0000909	17	−0.0001198	17	−0.0001342	17	−0.0001318
BEFORE UNLOADED	111	−0.0001699	200	−0.0001762	400	−0.0001752	100	−0.0001958	200	−0.0002202	400	−0.0002157
ELASTIC RECOVERY	90	−0.0001446	200	−0.0001539	400	−0.0001536	90	−0.0001590	200	−0.0001852	400	−0.0001813
ELASTIC AFTER EFFECT	104	−0.0001381	214	−0.0001484	414	−0.0001507	104	−0.0001501	214	−0.0001782	414	−0.0001766

DYS = Age of specimen in days; ΔL = deformation of specimen in inches per inch.

SECANT MODULUS OF ELASTICITY

		State of Stress		
		Low	Intermediate	High
CONCRETE	*Initial Load*			
	Stress change, psi	940	1550	2390
	E, psi	3.73×10^6	3.21×10^6	2.96×10^6
	Reload			
	Stress change psi	165	325	579
	E, psi	4.72×10^6	3.81×10^6	4.52×10^6
	increase, per cent	27	19	53
GUNITE	*Initial Load*			
	Stress change, psi	790*	1483	2425
	E, psi	4.29×10^6	2.65×10^6	3.07×10^6
	Reload			
	Stress change, psi	265	359	409
	E, psi	3.27×10^6	3.23×10^6	4.51×10^6
	increase, per cent	-24*	22	47
	TANGENT MODULUS OF ELASTICITY			
CONCRETE		4.88×10^6	5.95×10^6	4.78×10^6
GUNITE		5.75×10^6	5.74×10^6	6.16×10^6

*Initial load is 126 psi less than reload stress.

SECANT MODULUS OF ELASTICITY

State of Stress	Load Procedure	CONCRETE			GUNITE		
		Age days	Stress Change psi	E psi No. $\times 10^6$	Age days	Stress Change psi	E psi No. $\times 10^6$
Low	Reload	9	172	4.20	9	250	3.39
	Removed	104	336	3.68	105	133	3.58
	Reload	9	163	2.50	9	240	3.47
	Removed	200	386	4.96	200	412	3.32
	Reload	9	176	4.09	9	253	3.44
	Removed	400	276	3.33	400	381	3.07
Inter-mediate	Reload	12	311	3.45	12	389	2.47
	Removed	94	563	3.27	111	734	2.89
	Reload	12	342	4.37	12	295	2.94
	Removed	200	603	3.91	200	731	3.28
	Reload	12	321	4.19	12	318	3.25
	Removed	400	408	3.85	400	673	3.12
High	Reload	16	284	3.83	17	270	5.92
	Removed	111	911	3.72	100	1105	3.00
	Reload	16	450	4.22	17	387	5.69
	Removed	200	820	3.52	200	1002	2.87
	Reload	16	601	3.62	17	423	3.79
	Removed	400	675	3.54	400	1042	2.99

elasticity upon removal of the load is shown opposite; and, for the comparison, the earlier secant modulus of elasticity of the same three specimens when reloaded.

DISCUSSION OF RESULTS

Correlation of concrete and steel strains.

A test of the consistency of the data is a comparison of the change of length of the concrete Δl_s with that of the steel bars Δl_c. The two should be equal. Since the steel strains and modulus of elasticity are known, the steel stress and the total force in the bars can be ascertained, and also the concrete stresses. However, at the ends of the bars, the force is reduced as the threads at the bar ends mesh with the threads of the nuts. It is not known just how far each nut was tightened, so the total elongation Δl_s of the bars cannot be accurately computed.

The concrete strains were measured in the middle of the 24 in. length. It is probable that the strains at the ends were different due to the contact with the bearing plate, which inhibits free lateral deformation. Therefore, the true change in length Δl_c of the concrete or gunite specimen cannot be computed. During the loading and reloading of the specimens the nuts on the bars were tightened and the length of bar between nuts changed, including the consolidation of nuts on the bearing plates and the bearing plate on the concrete specimen. So the check of the accuracy of the readings is applied only to those readings after reloading. Adopting the arbitrary assumption that the measured steel strains times a length equal to the out-to-out distance between bearing plates plus one inch will give the steel deformation Δl_s, and that the concrete strain times 22.15 in. gives the concrete deformation Δl_c, a comparison of the two deformations subsequent to reloading at the ages of 100, 200 and 400 days is tabulated on the next page.

It would seem that the data are consistent, if one accepts some such uniform assumption of arbitrary lengths.

SHRINKAGE

The plot of shrinkage strains of non-stressed specimens in Fig. 2 has the usual shape of such data. The following shrinkage strains (next page) have been abstracted for comparison of the concrete and gunite shrinkages.

It is apparent that the gunite has much less shrinkage after the first 10 days of drying out.

PLASTIC FLOW

Reduction of prestress

The prestressing procedure adopted in these tests consisted of an initial prestress and a restoration of full prestress at 2 days thereafter. After

CHANGE IN LENGTH OF CONCRETE VERSUS STEEL BARS

State of Stress		100 days		200 days		400 days	
Low		Δl_c	Δl_s	Δl_c	Δl_s	Δl_c	Δl_s
l_c=22.15 in.	Concrete	0.01790	0.01822	0.01982	0.01965	0.02125	0.02116
l_s=27.5 in.	Gunite	0.01560	0.01561	0.1670	0.01644	0.01795	0.01788
Intermediate							
l_c=22.15 in.	Concrete	0.02012	0.01960	0.02250	0.02136	0.02385	0.02285
l_s=28.5 in.	Gunite	0.01634	0.01653	0.01810	0.01805	0.01972	0.01940
High							
l_c=22.15 in.	Concrete	0.02176	0.02060	0.02335	0.02232	0.02476	0.02305
l_s=30 in.	Gunite	0.01534	0.01623	0.01758	0.01830	0.01908	0.01983

SHRINKAGE STRAINS

	Age (days) After Zero Reading	Strain = $\frac{\text{in.}}{\text{in.}}$ = No. x 10^{-4}		Gunite Strain (as percent of Concrete Strain)
		Concrete	Gunite	
Expansion		+0.56	+0.56	100
Contraction	10	−1.35	−1.68	124
	20	−3.72	−2.91	78
	50	−6.47	−4.86	75
	100	−8.13	−5.82	72
	200	−8.20	−6.25	76
	400	−8.72	−6.54	75

reloading the adjustment of the nuts was not changed and the force in the bars and in the concrete decreased as shrinkage and plastic flow deformations occurred. This data will differ from many other tests performed with a constant force on the concrete. A salient fact obtained from these data is the marked decrease in the concrete and steel stresses as time elapsed. It indicates, as do other sources, that much of the effect of prestressing to such stresses as 30,000 to 35,000 psi is lost in a comparatively short time. The following tabulation shows the decrease in steel stresses during the time interval investigated.

The concrete and gunite stresses vary with the steel stress. While the final gunite stresses are less than 50 per cent of their prestress, they are

STEEL STRESS (psi)

	Concrete State of Stress			Gunite State of Stress		
	Low	Inter-mediate	High	Low	Inter-mediate	High
at reload	32,000	31,000	31,800	34,000	33,300	32,850
380 days later	10,460	8,260	9,020	14,980	13,920	14,050
per cent of reload	32.7	25.6	28.3	44.0	41.8	42.7

markedly higher than the corresponding concrete stresses. The latter are only ¼ to ⅓ their initial values.

PLASTIC FLOW COEFFICIENT

The additional strains in concrete or gunite that occur subsequent to reloading can be read from Fig. 3 or 4. At t days after reloading this strain amounts to e_t. The additional shrinkage strain e_s of the non-stressed specimens can also be read at t days after reloading from Figure 2. The term *plastic flow*, as used in this paper, will be defined as the difference $e_t - e_s = e_f$. This strain e_f includes flow strains due to the load and whatever difference there may be between shrinkage of non-stressed and stressed specimens. Within ordinary working stresses the plastic strain e_f is usually assumed to vary directly with the stress f_c, so that $e_f = f_c c$, where c is a flow coefficient that is a function of the time t. At some time t the stress in the concrete is f_c; a day or so later at the age $(t + \Delta t)$ the stress in the concrete has decreased an amount Δf_c. This decrease in stress corresponds to a tensile stress Δf_c added to the compression stress f_c to produce a resultant stress $(f_c - \Delta f_c)$. The immediate elastic effect of the change Δf_c is an increase of length whose strain is $\frac{\Delta f_c}{E_c}$, where E_c is the modulus of elasticity of the stressed specimen at the age t. In addition, during the time Δt there is plastic flow amounting to a strain $f_c(\Delta c)$, where Δc is the increase of the flow coefficient c in the time Δt. This is a compressive strain. The data indicate that the resultant of elastic and plastic changes in time Δt is an increase of the compressive strain Δe_f. Therefore:

$$\Delta e_f = f_c\,(\Delta c) - \frac{\Delta f_c}{E_c}$$

$$\text{or } \Delta_c = \frac{\Delta e_f + \frac{\Delta f_c}{E_c}}{f_c}$$

By use of the test data, f_c, Δf_c, and Δe_f can be computed. The actual value of the modulus of elasticity E_c of the stressed specimens at any age t is not known. The discussion on page 238 gives a few values of secant moduli for specimens unloaded at approximately 100, 200 and 400 days. The data indicate that E_c may be greater or less than the value at the time of reloading, usually less. The following tabulation of the flow coefficient c has been made assuming an average value of $E_c = 4.35 \times 10^6$ psi for concrete specimens and $E_c = 3.67 \times 10^6$ psi for the gunite. These values are the average E_c at the time of reloading. The value of the flow coefficient c was obtained by summing up values of Δc for a time interval of 5 days after reloading followed by 10 day intervals for the entire range of the tests.

Age (days) after reloading	Concrete			Gunite		
	State of Stress			State of Stress		
	Low	Intermediate	High	Low	Intermediate	High
50	33.4	49.1	47.7	35.9	38.2	31.8
100	45.2	63.9	55.0	46.0	44.6	39.7
200	57.9	71.8	62.9	50.0	53.8	45.0
380	64.1	79.3	67.3	57.7	60.8	50.3

If E_c were accurately determined at each age and the method of analysis were correct, the flow coefficient should be independent of the state of stress and all values of c at the age t should be the same. The results given above indicate that this is roughly true, since for both concrete and gunite the low and high stress values check fairly well. For both materials the intermediate values are greater than either low or high stressed specimens. The tabulation emphasizes again the fact that the plastic flow is less for the gunite than for the concrete, as is also true for the shrinkage strains of the unstressed specimens.

SUMMARY

The purpose of prestressing of concrete is to produce initial compressive stresses in the concrete that may balance tensile stresses caused by application of other loads. The tests described in this report apply to concrete members with a uniform prestress, such as pipes, tanks and columns. These tests indicate that moderate prestresses of 30,000 to 35,000 psi do not fulfill the object of prestressing since shrinkage and flow will reduce this prestress to less than one-half its original value for gunite and to ⅓ or ¼ the original value for concrete. That there is a decrease

has already been understood, but the magnitude of these reductions has not been generally known. The solution has been to employ high strength steel wires prestressed to 100,000 or 150,000 psi. A reduction of 20,000 to 30,000 psi due to shrinkage and flow still leaves a satisfactory prestress.

Unstressed specimens stored at 50 percent relative humidity had shrinkage strains at the end of 1 year of 8.7×10^{-4} for concrete and 6.5×10^{-4} for the gunite, the gunite shrinkage being about 75 percent of the concrete. After the second application of the full prestress to the loaded specimen (reload) the subsequent strains of the loaded specimens were greater than the corresponding shrinkage of the unloaded specimens. The difference, called plastic flow, is roughly proportional to the stress applied. Subsequent to reloading the shrinkage strains were a considerable part of the total strains of the loaded specimens. At 1 year for the different stress conditions they varied from 83 percent for low stress, 64 percent for intermediate, to 50 percent for the high stress for the concrete specimens. For the gunite the ratio of shrinkage to total deformation was 76 percent at low stress, 59 percent for intermediate, and 53 percent for high stress.

ACKNOWLEDGMENT

Grateful acknowledgment is made to the Preload Company of Boston which furnished financial assistance for these tests and to Messrs. P. D. Johnson, R. B. Snow and A. J. O'Neill who did the most of the experimental work.

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Discussion of a paper by Howard R. Staley and Dean Peabody, Jr.:

Shrinkage and Plastic Flow Prestressed Concrete*

By PAUL WILLIAM ABELES and AUTHORS

By PAUL WILLIAM ABELES†

Any new investigations on shrinkage and plastic flow in prestressed reinforced concrete are of interest. The purpose of the tests described in this paper was to study the "behavior of prestressed reinforced concrete specimens (for which the load is not constant but varies with time and condition of exposure and use) under conditions that are probably the worst that could be conceived for field usage."

The steel stresses are limited to 31,800 to 34,000 psi which has been known to be too low to insure an effective pre-compression of appreciable magnitude, even if "post-stretching" is applied whereby the initial losses due to shrinkage and elastic strain (shortening of the concrete) do not occur. The conclusion drawn from the tests is therefore, as stated, only a further confirmation of established knowledge. However, the tests show an extraordinarily high shrinkage, much higher than known before. The maximum unit shortening due to shrinkage of high strength concrete is generally assumed to be 250×10^{-6}, which value may increase to a maximum of 500×10^{-6} for a concrete of great shrinkage.

It is surprising to note that a shrinkage of 875×10^{-6} for concrete has been ascertained after 400 days, and it would have been interesting to learn also the magnitude of the plastic flow under sustained pressure for the 3 different compressions. However, there seems to be some discrepancy between the measured unit shrinkage and coefficient for plastic flow on the one hand and the measured loss of the tensioning stress in the steel on the other hand. The latter amounts, according to the table p. 241, for the 3 stresses of the concrete specimens to 21,540, 22,740 and 22,780 psi, corresponding to a concrete strain of 735, 801 and 802×10^{-6} respectively ($E_s = 29.3 \times 10^6$ for low stress, bars $\frac{3}{8}$ in. dia. and $E_s = 28.4 \times 10^6$ psi for intermediate and high stresses, bars $\frac{1}{2}$ in. dia.), and

*ACI JOURNAL, Jan. 1946; *Proceedings* V. 42, p. 229.
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thus compare well with known values. However, it is different with the concrete strain. When investigating, for example, only concrete specimens of low stress, the shrinkage alone amounts to approx. 800 x 10^{-6} (according to Fig. 2) and the entire strain to approximately 1450 x 10^{-6} (according to Fig. 3), which latter value is reduced, due to elastic strain, by 232 to 310 x 10^{-6} (for E_c = 3 to 4 x 10^6 and low stress of 930 psi), resulting in 1140 to 1228 x 10^{-6}.

The writer has suggested[1]* to consider maximum losses of 0.001 E_s, due to shrinkage, Δp_s, and plastic flow, Δp_p, i.e. 25,000 to 30,000 psi for E_s = 25 to 30 x 10^6 psi. Schorer recommends tentatively on the basis of tests the following losses: $\Delta p_s + \Delta p_p = 15{,}000 + 7.5\ f_c$, f_c being the effective pre-compression of the concrete. Other values are stated in the British Standard Specification for Sleeper (Ties)[3]: $\Delta p_s = 0.0003\ E_s$ and $\Delta p_p = 0.3\ E_s \times 10^{-6}\ f_c$, which amount for E_s = 25 to 30 x 10^6 to Δp_s = 7,500 to 9,000 psi and $\Delta p_p = 7.5$ to $9\ f_c$. All these values relate to "pre-stretching" where the reinforcement is bonded to the concrete and the entire shrinkage has to be considered. With "post-stretching," however, tensioning can be carried out after a considerable part of the shrinkage has already taken place; consequently a reduced value of 15,000 to 25,000 psi may be considered for this process. It is impossible to derive values corresponding to these quantities from the test results published in the present paper. It would be interesting to learn how the authors account for this discrepancy. There is still a further point. For the same concrete, different maximum flow coefficients are published; they are greater for an intermediate stress than both for a high or low concrete stress. Also in that respect some clarification would be welcome.

It has been realized, as the authors point out, that the use of high strength steel wires, tensioned to 100,000 to 150,000 psi is the solution for prestressed reinforced concrete. Although this was recognized as early as 1925 to 1928, when suggested by Messrs. Dill and Freyssinet respectively[1], it was only recently that Prof. G. Magnel[4] discovered also that the tensioned wires undergo plastic flow; this means that the loss Δp_p consists of 2 parts, one caused by plastic flow of the concrete and the other by that of the wires, which latter according to Professor Magnel amounts to 5 to 6 per cent. He has come to the conclusion that a considerable part of plastic flow of steel can be avoided, when, at tensioning, the initial prestress is increased, say by 10 per cent, and the increased stress is maintained for some minutes. In this connection it may be mentioned that Professor Magnel has developed a new method of "post-stretching" which was successfully applied to railway underbridges built after the liberation of Belgium in 1945.

*See references at end of text.

It appears to be important to carry out further investigations on shrinkage and plastic flow of concrete and steel for various qualities and under substantially sustained effective pre-compression as it occurs when high tensioning stresses are applied. Such further research would be the more important, in view of the great possibilities of prestressed reinforced concrete. One specific feature of effectively prestressed reinforced concrete, namely its extraordinary resilience was unknown for a long time because in fully prestressed structures, which alone were advocated, total absence of cracks is guaranteed, and consequently, it was not of great interest to investigate the behavior under increasing load up to failure. Comparative tests, carried out in March 1946, on reinforced concrete ties some of which were in use in the main line, London-Edinburgh, have proved the extraordinary standard of prestressed reinforced concrete.

In any case, as long as detailed research data are not available, maximum losses due to shrinkage and plastic flow of 25,000 to 30,000 psi for "pre-stretching" and 15,000 to 25,000 psi for "post-stretching" ought to be taken into account.

REFERENCES

(1) "Fully and Partly Prestressed Reinforced Concrete" by P. W. Abeles, ACI JOURNAL, V. 16, No. 3, January 1945.

(2) "Prestressed Concrete, Design Principles and Reinforcing Units" by H. Schorer, ACI JOURNAL, V. 14 No. 6, June 1943.

(3) War Emergency British Standard No. 986/1945 "Concrete Railway Sleepers" (Ties), British Standard Institution.

(4) "Prestressed Concrete: Some new Developments" by G. Magnel, *Concrete & Constr. Eng.*, Nov. and Dec. 1945 and Jan. 1946.

AUTHORS' CLOSURE

The laboratory tests reported in this paper were made to procure data on the bahavior of prestressed gunite and concretes to be used in certain war time construction. These projects in the field were planned for the low prestress of 32,000 to 34,000 psi. The tests differ from most previous laboratory results in that the prestress was not maintained at a constant value during the time of test, but was applied at an early age in two adjustments and allowed to decrease as shrinkage and plastic flow might dictate.

In the third paragraph of his discussion Mr. Abeles finds an apparent discrepancy in the data. He correctly reports from the table on p. 241 the decrease of steel stresses, after reloading is effected, as 21,540, 22,740 and 22,780 psi for the concrete specimens. The corresponding *steel* strains are 735, 801 and 802 x 10^{-6}. However, the corresponding *concrete*

strains are not equal to these values as the lengths of steel bars and concrete specimens are not the same. The concrete and steel strains are related by the fact that the total change in length of the steel bars and of the concrete specimens must be the same, as discussed on p. 239 and in the table at the top of p. 240. Using the arbitrary lengths of that table and the actual data on Table 2, the ratio of concrete to steel strains for the concrete specimens should be:

	from p. 240	from Table 2
low stress	$\frac{27.5}{22.15} = 1.24$	$\frac{1474-514}{1118-348} = 1.25$
intermediate	$\frac{28.5}{22.15} = 1.29$	$\frac{1992-914}{1090-289} = 1.35$
high stress	$\frac{30}{22.15} = 1.35$	$\frac{2570-1451}{1120-319} = 1.40$

Thus, the actual concrete strains during the decrease of steel stress from the time of reloading to 400 days are 960, 1078 and 1119 x 10^{-6} instead of the values of 735, 801, and 802 x 10^{-6} that Mr. Abeles reports. These concrete strains occur only during the time of decrease of steel stress so that the *entire* strain of 1450 x 10^{-6} for the low stressed concrete reported later in the paragraph is irrelevant.

Mr. Abeles has also assembled recommendations for the reduction in steel stresses proposed by himself and by Mr. Schorer, as well as by the British Standard Specification for Sleepers. For a modulus of elasticity of the steel $E_s = 29 \times 10^6$ psi the Abeles reduction of steel stress is 29,000 psi, the Schorer reduction from 22,000 to 33,000 psi and the British Standard from 17,000 to 29,000 psi. As quoted above, the steel stress reductions in this paper are 21,540 to 22,780 psi.

The authors heartily agree with Mr. Abeles that it is important to have further investigation on shrinkage and plastic flow employing much higher prestresses.

[To facilitate selective distribution, **separate prints** of this title (42-11) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this report (copies in triplicate) should reach the Institute not later than April 1, 1946.]

Title 42-11 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

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Proposed Minimum Standard Requirements for Precast Concrete Floor Units*

REPORTED BY ACI COMMITTEE 711

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GENERAL

10. Scope or limits

(*a*) These minimum standard requirements for precast concrete floor units, are to be used as a supplement to the ACI "Building Regulations for Reinforced Concrete" (ACI 318-41)†.

(*b*) With respect to design for strength, i.e., for bending moment, bond and shear stresses, all the types referred to in 10 (d) are to be designed in accord with standard reinforced concrete theory and in accord with (ACI 318-41), except that with respect to cover, there is in some cases departure therefrom justified by the greater refinement in the finished product, both as to dimensions and to quality, when made by factory methods and with factory control. See Section 11.

(*c*) Section 103(*a*), ACI 318-41‡, recognizes and makes provision for special systems of reinforced concrete. With reference to precast floors its provisions may be invoked where necessary by the manufacturer, architect, engineer, building inspector, builder or owner, wherever this report is silent.

*The Committee's report as published ACI JOURNAL, February 1944, Proceedings V. 40, p. 305, is now published with revisions as released by the ACI Standards Committee for presentation to the 1946 Convention on a motion for adoption as an ACI Standard.
†Reference is hereafter referred to as ACI 318-41.
‡See appendix hereto.

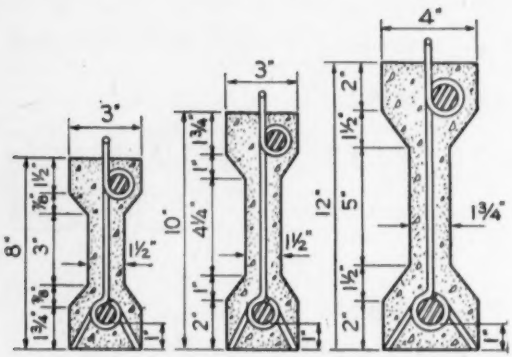
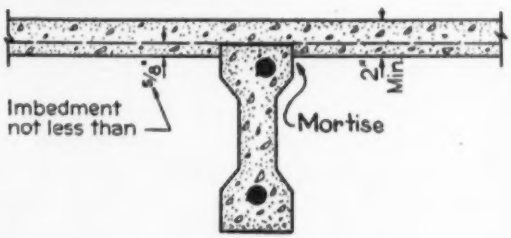


Fig. 1

Fig. 2 — Note: The mortise shall be not less than ½ in.



(*d*) Two distinct types are now manufactured:

1. I-beam type, with either cast in place or precast slab. (Fig. 1)

See section 50 for definitions. When the slab is cast in place as shown in Fig. 2 the result is a Tee beam and may be computed as such.

2. Hollow core type (Fig. 5)

Others are of channel shaped cross section often used as roof slabs, and the orthodox rectangular beam. Standard reinforced concrete theory is applicable to all.

11. Concrete protection for reinforcement

(*a*) Precast floor and roof units made of high quality factory controlled concrete may, when used in locations protected from the weather or moisture and with minimum fire hazards, be approved with ⅝-in. concrete cover for the reinforcing provided, however, that the concrete cover in all cases shall be at least equal to the diameter of round bars and one and one-half times the side dimension of square bars, and provided that to insure exact final location to the steel, positive and rigid devices for that purpose are employed in the manufacturing process. When the precast-members are exposed to weather, moisture or fire hazard the protective cover shall be increased to conform with section 507, ACI 318-41.†

†See appendix hereto.

MATERIALS

20. Cement

(*a*) High-early strength concrete as produced with Type III portland cement or with Type I portland cement and accelerated curing is recommended. Portland cement shall conform to the "Standard Specifications for Portland Cement" (A.S.T.M. Serial Designation C150-42) and shall be Type I or Type III.

21. Aggregate

(*a*) Concrete aggregates shall conform to the "Standard Specifications for Concrete Aggregates" (ASTM Serial Designation: C33-44), provided, however, that aggregates which have been shown by test or actual service to produce concrete of the required strength, durability, water-tightness, fire-resistance, and wearing qualities may be used under Section 302(*a*) Method 2, ACI 318-41* where authorized by the Commissioner of Buildings.

(*b*) The maximum size of the aggregate for precast joist shall not be larger than one-third of the narrowest dimension between sides of the forms of the member in which the unit is cast nor larger than three-fourths of the minimum clear spacing between reinforcing bars and sides of the forms except that where the concrete is placed by means of high frequency vibration the maximum size of the aggregate shall not be larger than one-half the narrowest dimension between sides of the forms.

(*c*) Aggregate for floor slabs shall conform to Section 21 (a) and in addition the combined aggregate shall be so graded from fine to coarse that not less than one-half nor more than two-thirds by weight of the total, based on dry materials, is retained on the No. 4 standard sieve, except that these proportions do not necessarily apply to light weight aggregates. The maximum size shall not exceed one-third the thickness of the slab.

22. Steel

(*a*) In the unprestressed types, the steel in the joist or floor units shall be intermediate grades Billet-Steel Concrete Reinforcement Bars (ASTM Serial Designation. A15-39) or Rail Steel Concrete Reinforcement Bars (ASTM Serial Designation: A16-39) or cold drawn steel wire for concrete reinforcement (ASTM Serial Designation: A82-34).

(*b*) Prestressed steel may be used in any of the types mentioned in section (10(d). When used, computations for stresses, moments and allowable loads shall be in accord with the theory outlined in "Prestressed Concrete, Design Principles and Reinforcing Units", by Herman Schorer† or in a more condensed article by the same author in Reinforced Concrete No. 6 (Portland Cement Assn.).

*See appendix hereto.
†ACI JOURNAL, June 1943; Proceedings V. 39.

23. Strength of concrete

(*a*) Concrete for floor units made of sand and gravel, crushed stone, slag or other heavy aggregate and of a span of 12 ft. or more shall have a compressive strength of not less than 3750 psi at 28 days when tested in accordance with the applicable current standards of the A.S.T.M.

(*b*) For roof slabs or for floor units made of light weight aggregate lower compressive strengths may be permitted where the unit stresses used in design for strength and bond will satisfy the requirements of paragraph 24(a) of these standards.

24. Unit stresses in concrete and reinforcement

(*a*) The allowable *design* stresses in the concrete *shall* conform to the requirements set forth in Section 305 (a) and Table 305(a), ACI 318-41.†

(*b*) The allowable stresses in the steel shall conform to the requirements set forth in Section 306(a) and 306(b) ACI 318-41†.

MANUFACTURE

30. Workmanship

(*a*) The finished product shall be free of honeycomb or rock pockets. The mix, the gradation of the aggregate and the workability shall be such as to insure complete filling of the form and continuous intimate bond between the concrete and all steel. To assist in attaining the latter, vibration is recommended, but any method which will meet the stated requirements and with the strength as required in unit beams or slabs or in the finished floor, is acceptable.

(*b*) Handling and conveying before curing shall be reduced to a minimum. Machinery for this purpose should be so designed that the unit will not be subject to bending or shock which will produce incipient cracks, broken edges or corners.

31. Curing

(*a*) The minimum amount of curing of precast units shall consist in keeping the concrete moist for at least 7 days, if made of normal portland cement and for at least 3 days if made of high early strength cement. For each decrement of 5 degrees below 70 F in the average curing temperature these curing periods shall be increased by four days for units made of normal portland cement and by two days for units made of high early strength cement. See Table 1. The average curing temperature in no case shall be less than 50 F.

(*b*) Curing by high pressure steam, steam vapor, or other accepted processes may be employed to accelerate the hardening of the concrete and to reduce the time of curing provided, however, the compressive

†See appendix hereto.

TABLE 1 (sec 31a) — MINIMUM CURING TIME IN DAYS IN MOIST ATMOSPHERE

Cement	Temperature in Degrees F				
	70	65	60	55	50
Normal Portland.....	7	11	15	19	23
High Early Strength.....	3	5	7	9	11

strength of the concrete is at least equal to that obtained with the curing specified in Section 31a and that the 28-day strength meets the requirements of Sec. 23.

32. Identification and marking

(*a*) All joist, beams, girders and other floor units shall show some mark plainly indicating the top of the unit and the size of the bending moment reinforcement. This mark or symbol shall indicate the length, size and type of reinforcing and carrying capacity of the unit, and shall be shown on the placing plans.

33. Transportation

(*a*) After curing, units shall be so stored, stacked, loaded and transported, unloaded and placed, that no transverse or longitudinal cracks will develop. Adequate instructions should be given to handlers and insofar as possible, only experienced men should be put in charge of this phase of the work.

(*b*) To insure the eventual placement of the units in the structure without cracks, the handling, whether manually or in slings or cradles, shall be done in such a manner that bending about either the vertical or horizontal axis of the cross-section will be reduced to a minimum.

TESTS

40. Beams and floors

(*a*) Where the individual unit is tested as a simple beam, it shall sustain without complete failure*, a uniformly distributed load of at least 2.25 times the design live load based on allowable stresses in bending moment and shear as given in Section and Table 305(*a*) and Section 306(*a*), ACI 318-41†

(*b*) When field tests are made they shall be made as required by and shall meet the requirements of Section 202, ACI 318-41†) making use of notation used in Section 200, ACI 318-41.†

*Failure in Section 40 is defined as, any behavior of the beam under load which indicates that the yield point of the steel has been exceeded, or that cracking of the concrete is such that it would not be permitted in a structure in regular service.

†See appendix hereto.

I-BEAM TYPE JOISTS

50. Definitions

(*a*) Floors made of precast joists mortised or embedded into a monolithic floor placed or poured on the job, producing a T-beam are called *Precast joist cast-in-place concrete slab floors*, and the resisting moment may be computed as if the joist and slab form a T-beam. See Section 54.

(*b*) Floors made of precast joists over which precast slabs are laid and bonded to produce T-beam action are called *precast joist* and *slab* concrete floors.

(*c*) Floors made of precast joists over which precast slabs are laid for flooring and *not* bonded to the joist to produce T-beam action are called *independent precast joist* and *slab* concrete floors.

51. Sections

(*a*) The most commonly made joists of I-beam section are as shown in Fig. 1. Other sizes and shapes meeting the regulations of ACI 318-41 as to resistance to bending moment, shear, deflection and bearing, may be used.

(*b*) Since the shear and bending moment resistances are based on nominal dimensions as well as on area of the steel and allowable working stresses in concrete and steel, the following tolerances shall not be exceeded: plus or minus $\frac{1}{8}$-in. as to width and height, and plus or minus $\frac{1}{2}$ in. as to length.

52. Floor slab thickness

(*a*) The recommended minimum thickness of *cast-in-place* reinforced concrete floor slabs with joist heads embedded not less than $\frac{1}{2}$ in. and with joist spacing less than 30 in., is 2 in. For joist spacing of 30 to 36 in. the minimum thickness of *slab* concrete floors should be $2\frac{1}{2}$ in. Greater thickness may be required where unusual loads or spans are encountered. The required thickness of slabs spanning more than 36 in. shall be determined by accepted design methods.

(*b*) The recommended minimum thickness of precast slab to be used with precast joists is 2 in. with joist spacing up to 30 in. and $2\frac{1}{2}$ in. with joist spacing from 30 in. to 36 in. In the case of slabs of ribbed- or channel-section, the thickness requirement applies to the portion thereof containing the tensile reinforcement.

53. Extra or concentrated loads

(*a*) Where the floor supports partition walls parallel to the joist or where loads heavier than the uniform load for which the floor is designed are known to be expected, joists may be placed side by side, with flanges touching, but under such conditions the joists and cast-in-place floor slabs are not to be considered as a T-beam unless the shear reinforcing loops in the joists extend into the slab.

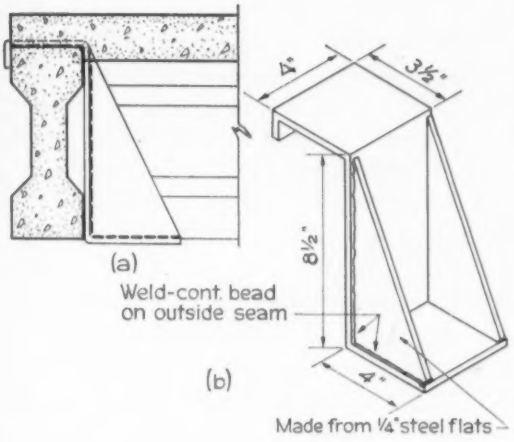
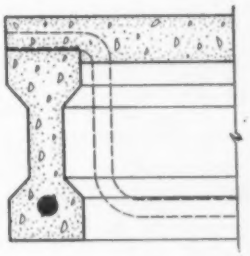


Fig. 3—Details of joist hangers

Fig. 4—Tension bar hanger inserted in joist as cast



(*b*) Where multiple joists are used, their strength shall be at least that of a single beam, multiplied by the number used.

54. Design

(*a*) Floors laid as defined in 50(*a*) and 50(*b*) of these standards may be designed as T-beams, where joists are supported at sufficient intervals to take out the sag while the cast-in-place or precast slab is being laid, the support being left in place until the concrete has hardened. Under this condition the dead load is considered to be the weight of the floor per joist plus the weight of the joist.*

(*b*) The resistance to longitudinal shear between floor and joist where the joists are embedded ½ in. may be taken as equal to the allowable shear stress for beams with no web reinforcement, but with special anchorage of longitudinal steel. Table 305(*a*), ACI 318-41†.

(*c*) Where ends of joists cannot be rested on walls, as at stair wells, etc., metal joist hangers made as in Fig. 3 may be used to provide end support, or a preformed tension bar hanger may be inserted in the joist at the time of casting (Fig. 4).

*For further details as to design for Type 1 joists, the Portland Cement Association pahmphlet "How to Design and Build Precast Joist Concrete Floors" is at present the most complete treatise.
†See appendix hereto.

55. Holes in web

(*a*) Because they reduce the shearing resistance, holes in the web shall be reduced to a minimum. Where found necessary they should be located as near the center of the beam as possible or at location of minimum shear. They shall be cast when the beam is made or drilled on the job (not punched) and be not more than 2 in. in diameter. No holes should be made by any mechanic on a job except after approval by and under the supervision of the architect or engineer.

56. Installation and construction details

(*a*) On every job there will be a need for a joist setting plan; prepared by an architect or an engineer and approved by the manufacturer. Only in this way will the owner be assured of unquestionable results. Working stresses based on maximum strength of the materials used, shall be as provided in Section 305(*a*) and Table 305(*a*), ACI 318-41*, and shall be given the architect or engineer by the manufacturer. Shears and bending moments will be properly taken into account by the architect or engineer and accepted by the manufacturer.†

(*b*) There is a need for standardization of many installation and construction details. This does not mean that innovations should be prohibited or frowned upon, but rather that an acceptable practice in handling and setting, leveling, shoring of joists, placing forms for floors, reinforcement for floors, conduits, bulkheads, stairwells, partition bearing joist, etc., should be approved by the architect, engineer and manufacturer and the building contractor informed thereof by properly drawn plans and through the supervision of the architect or engineer.

HOLLOW CORE TYPE JOISTS

60. Definitions

(*a*) Floors made of precast concrete units in which some portion of the cross section between top, bottom, and sides is left out at the time of casting for purposes of reducing dead load and quantity of material used in their manufacture are called Hollow-Core-Precast Floors.

61. Sections

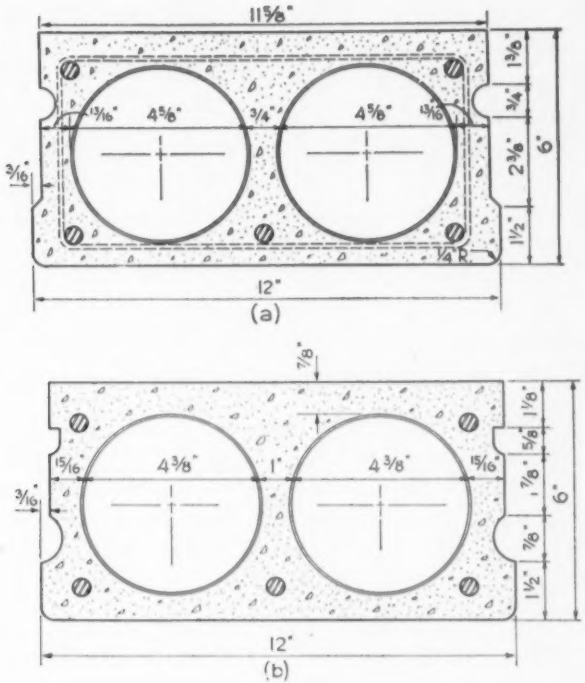
(*a*) Sections of hollow units as shown in Fig. 5 are acceptable.

(*b*) Any other sections which will provide proper protection for the steel reinforcement, and strength sufficient for handling and for carrying the loads for which they are designed and which meet all other requirements of this report will be acceptable, provided the computations for carrying capacity have been verified by a recognized testing laboratory.

*See appendix hereto.

†The manufacturer's supervision over the placing of the reinforcement and over the factory treatment of the joist from planning the mixture to delivery on the job, will be stricter, where he is provided with data on the maximum shear and bending moment stresses as computed by the architect or engineer.

Fig. 5



62. Design

(*a*) Resisting moments and shear resistance of hollow core units shall be computed by the standard formulas and methods. Allowable unit stresses in concrete shall conform to the requirements of Section 305(a) and Table 305(a) ACI 318-41.*

(*b*) The allowable stresses in the steel shall conform to the requirements set forth in Section 306(a) and 306(b) ACI 318-41.*

63. Special conditions—openings known in advance of construction

(*a*) At stairways or other openings when no wall or girder bearing is available for one end of a floor unit, the long dimension of the opening shall be parallel to the length of the unit. Specially designed reinforced headers or curbs at the short side of such openings shall transfer their dead and live load to the longitudinal units on the side of the opening by devices or means satisfactory to the architect or engineer. Units adjacent to the side of such openings shall be designed to carry the reaction of the headers in addition to their own specified dead and live load.

(*b*) The requirements of unusual conditions, such as those outlined in 63(*a*) or others, often may be met by making use of special units,

*See appendix hereto.

some of which are wider than the standard, some deeper and some more heavily reinforced.

(*c*) Holes in the bottom or ceiling sides of units, for conduit or for hangers should be located below the hollow portion of the unit and should be cast at the time of manufacture or drilled under the supervision of the architect or engineer.

(*d*) Channeling in the top or floor side, except over the support is not permissible and channels placed over the supports shall not reduce the shear resistance below that allowable in this standard.

(*e*) Cutting of reinforcement for installation of pipes or conduit is permitted only upon approval of the architect or engineer and only after satisfying requirements specified in Section 64(*a*) (*b*) and (*c*).

64. Cutting of holes and channels

(*a*) No openings or channels not provided for in the structural design shall be made on the job without the specific approval of the engineer and in accord with his written, detailed instructions covering such work.

(*b*) In some cases the section of and reinforcement in the adjacent beam are such that when the span is taken into consideration the resistance to bending moment and shear is greater than that required by the live and dead loads called for by the building code or specifications. In such a case holes may be cut and curbed providing it is done in a manner to insure that the stresses on the transversely cut units will be transferred through the curb or longitudinal key to the adjoining units. In general, such cutting should be located near the quarter point of the span.

(*c*) Where holes are cut, the load normally carried by the cut units, and by the cutting transferred laterally to adjacent units may be considered to be uniformly distributed laterally for three units one foot wide on either side. With such assumption the computed stresses in the concrete may not esceed $0.45 f'_c$ nor in any case 1500 psi compression; $0.02f'_c$ (for reinforcement without special anchorage) with a maximum allowable unit stress of 75 psi or $0.03\ f'_c$ (for reinforcement with special anchorage) with a maximum allowable unit stress of 112.5 psi in shear for units without stirrups; or 20,000 psi tension in the reinforcing steel.

(*d*) Holes in the bottom or ceiling side for conduit or for hangers should be below the hollow portion of the unit; should not be less than 1¾ in. from the longitudinal reinforcement, and may be either cast at the time of manufacture or drilled under the supervision of the architect or engineer. Hangers may be placed in the joints between the units before they are grouted.

(*e*) There shall be no channeling in the top or floor side except over the support. Channels cut over the supports are permissable only when they do not reduce the shear resistance below that allowable in this standard, and must be approved by the architect or engineer.

(*f*) Cutting of reinforcement for installation of pipes or conduit is permissible only with approval of the architect or engineer and only after satisfying requirements specified in Section 64 (*a*) (*b*) and (*c*).

65. Installation and construction details

(*a*) An erection or unit setting plan shall be prepared by the architect or engineer and approved by the manufacturer for each job. To provide properly for shear and bending moment stresses, the plan shall indicate all openings, stairways, etc., together with the location of points, if any, where loads are in excess of the general floor loading.

APPENDIX

(Excerpts from Standard Building Regulations for Reinforced Concrete—ACI 318-41)

103. Special systems of reinforced concrete

(*a*) The sponsors of any system of reinforced concrete which has been in successful use, or the adequacy of which has been shown by test, and the design of which is either in conflict with, or not covered by these regulations shall have the right to present the data on which their design is based to a "Board of Examiners for Special Construction" appointed by the Commissioner of Buildings. This Board shall be composed of competent engineers, architects and builders, and shall have the authority to investigate the data so submitted and to formulate rules governing the design and construction of such systems. These rules when approved by the Commissioner of Buildings shall be of the same force and effect as the provisions of this code.

200. Notation

D = Deflection of a floor member under load test.
L = Span of member under load test.
t = The total thickness or depth of a member under load test.

202. Load Tests

(*a*) When a load test is required, the member or portion of the structure under consideration shall be subject to a superimposed load equal to one and one-half times the live load plus one-half of the dead load. This load shall be left in position for a period of twenty-four hours before removal. If, during the test, or upon removal of the load, the member or portion of the structure shows evident failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made; or, where lawful, a lower rating shall be established. The structure shall be considered to have passed the test if the maximum deflection at the end of the twenty-four hour period does not exceed the value of D as given in the following:

$$D = \frac{.001\,L^2}{12\,t} \qquad (1)$$

all terms expressed in the same units.

If the deflection exceeds the value of D as given in formula (1), the construction shall be considered to have passed the test if within twenty-four hours after the removal of the load the member or portion of the structure shows a recovery of at least seventy-five percent of the observed deflection.

302. Determination of strength-quality of materials

(*a*) The determination of the proportions of cement, aggregate and water to attain the required strengths shall be made by one of the following methods:

Method 1—Concrete made from average materials:

When no preliminary tests of the materials to be used are made, the water-content per sack of cement shall not exceed the values in Table 302(*a*). Method 2 shall be employed when artificial aggregates or admixtures are used.

TABLE 302(a)—ASSUMED STRENGTH OF CONCRETE MIXTURES

Water-Content U. S. Gallons Per 94-lb. Sack of Cement	Assumed Compressive Strength at 28 Days—psi
7½	2000
6¾	2500
6	3000
5	3750

NOTE—In interpreting this table, surface water carried by the aggregate must be included as part of the mixing water in computing the water-content.

Method 2—Controlled Concrete:

Water-content other than shown in Table 302(*a*) may be used provided that the strength-quality of the concrete proposed for use in the structure shall be established by tests which shall be made in advance of the beginning of operations, using the consistencies suitable for the work and in accordance with the "Standard Method of Making Compression Tests of Concrete" (A.S.T.M. Serial Designation: C39-39). A curve representing the relation between the water-content and the average 28-day compressive strength or earlier strength at which the concrete is to receive its full working load, shall be established for a range of values including all the compressive strengths called for on the plans.

The curve shall be established by at least three points, each point representing average values from at least four test specimens. The maximum allowable water-content for the concrete for the structure shall be as determined from this curve and shall correspond to a strength which is fifteen percent greater than that called for on the plans. No substitutions shall be made in the materials used on the work without additional tests in accordance herewith to show that the quality of the concrete is satisfactory.

305. Allowable unit stresses in concrete

(*a*) The unit stresses in pounds per square inch on concrete to be used in the design shall not exceed the values of Table 305(*a*) where f'_c equals the minimum specified ultimate compressive strength at 28 days, or at the earlier age at which the concrete may be expected to receive its full load.

306. Allowable unit stresses in reinforcement

Unless otherwise provided in these Regulations, steel for concrete reinforcement shall not be stressed in excess of the following limits:

(*a*) *Tension*

(f_s = Tensile unit stress in longitudinal reinforcement)
and (f_s = Tensile unit stress in web reinforcement)

20,000 psi for Rail-Steel Concrete Reinforcement Bars, Billet-Steel Concrete Reinforcement Bars (of intermediate and hard grades), Axle-Steel Concrete Reinforcement Bars (of intermediate and hard grades), and Cold-Drawn Steel Wire for Concrete Reinforcement.

18,000 psi for Billet-Steel Concrete Reinforcement Bars (of structural grade), and Axle-Steel Concrete Reinforcement Bars (of structural grade).

507. Concrete protection for reinforcement

(*a*) The reinforcement of footings and other principal structural members in which the concrete is deposited against the ground shall have not less than three inches of concrete between it and the ground contact surface. If concrete surfaces after removal of the forms are to be exposed to the weather or be in contact with the ground, the reinforcement shall be protected with not less than two inches of concrete for bars more than ⅝ inch in diameter and one and one-half inches for bars ⅝ inch or less in diameter.

TABLE 305(a)—ALLOWABLE UNIT STRESSES IN CONCRETE

Description		Allowable Unit Stresses				
		For Any Strength of Concrete as Fixed by Test in Accordance with Section 302 $n = \frac{30000}{f'_c}$	When Strength of Concrete is Fixed by the Water-Content in Accordance with Section 302			
			f'_c = 2000 psi n = 15	f'_c = 2500 psi n = 12	f'_c = 3000 psi n = 10	f'_c = 3750 psi n = 8
Flexure: f_c						
Extreme fiber stress in compression	f_c	$0.45f'_c$	900	1125	1350	1688
Shear: v						
Beams with no web reinforcement and without special anchorage of longitudinal steel	v_c	$0.02f'_c$	40	50	60	75
Beams with no web reinforcement but with special anchorage of longitudinal steel	v_c	$0.03f'_c$	60	75	90	113
Beams with properly designed web reinforcement but without special anchorage of longitudinal steel	v	$0.06f'_c$	120	150	180	225
Beams with properly designed web reinforcement and with special anchorage of longitudinal steel	v	$0.12f'_c$	240	300	360	450
*Flat slabs at distance d from edge of column capital or drop panel	v_c	$0.03f'_c$	60	75	90	113
**Footings	v_c	$0.03f'_c$ but not to exceed 75 psi	60	75	75	75
‡Bond: u						
In beams and slabs and one-way footings:						
Plain bars	u	$0.04f'_c$ but not to exceed 160 psi	80	100	120	150
Deformed bars	u	$0.05f'_c$ but not to exceed 200 psi	100	125	150	188
In two-way footings:						
Plain bars (hooked)	u	$0.045f'_c$ but not to exceed 160 psi	90	113	135	160
Deformed bars (hooked)	u	$0.056f'_c$ but not to exceed 200 psi	112	140	168	200
Bearing: f_c						
On full area	f_c	$0.25f'_c$	500	625	750	938
On one-third area or less†	f_c	$0.375f'_c$	750	938	1125	1405

*See Section 807. **See Section 905 (*a*) and 808(*a*).

†The allowable bearing stress on an area greater than one-third but less than the full area shall be interpolated between the values given.

‡Where special anchorage is provided (see Section 903 (*a*)), one and one-half times these values in bond may be used in beams, slabs and one-way footings, but in no case to exceed 200 psi for plain bars and 250 psi for deformed bars. The values given for two-way footings include an allowance for special anchorage.

(*b*) The concrete protective covering for reinforcement at surfaces not exposed directly to the ground or weather shall be not less than three-fourths inch for slabs and walls; and not less than one and one-half inches for beams, girders and columns. In concrete joist floors in which the clear distance between joists is not more than thirty inches, the protection of metal reinforcement shall be at least three-fourths inch.

(*c*) If the code of which these regulations form a part specifies, as fire-protective covering of the reinforcement, thicknesses of concrete greater than those given in this section, then such greater thicknesses shall be used.

(*d*) Concrete protection for reinforcement shall in all cases be at least equal to the diameter of round bars, and one and one-half times the side dimension of square bars.

(*e*) Exposed reinforcement bars intended for bonding with future extensions shall be protected from corrosion by concrete or other adequate covering.

[**Separate prints** of this title (42-12) are currently available with the report of January 1944 at 25 cents each—quantity quotations on request. **Discussion** of this report as revised (copies in triplicate) should reach the Institute not later than April 1, 1946.]

Title 42-12 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
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Vol. 17 No. 3 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **January 1946**

Proposed Recommended Practice for the Construction of Concrete Farm Silos*

Reported by ACI Committee 714

WILLIAM W. GURNEY
Chairman

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WALTER BRASSERT
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CHARLES F. ROGERS
STANLEY WITZEL

AS HERE REVISED REPORT IS PRESENTED FOR ADOPTION AS A STANDARD

This constitutes formal notice to the membership of the Institute that the report of the Committee 714, William W. Gurney, Chairman, published p. 189 of the ACI JOURNAL January 1944 has been revised by the originating committee in the light of discussion p. 204-1 of the November 1944 JOURNAL *Supplement.*

Committee 714 now asks for the adoption of the report as a Standard of the Institute subject to the following revisions:

Under "2. General Design Recommendations," sub-section B1). "Footings" is revised to read as follows:

B. Footings 1) *Depth below ground line:* The distance from the ground line to the base of the footing should be at least 24 in. and not less than 36 in. in localities in which the ground freezes to a depth of two feet or more.

Page 192, of the report as originally published, the title of Table 3 is revised to read "Table 3—Hoop spacing for concrete stave silos *designed for silage with a moisture content not exceeding 75 percent,*" (the italicized words constituting an addition to the previous title).

*The report (published ACI JOURNAL, January, 1944; Proceedings V. 40, p. 189) as here revised and with some contemplated revisions of an editorial nature, has been released by the ACI Standards Committee for presentation to the Institute's 1946 Convention on a motion for adoption as an ACI Standard.

In section "3. Materials," sub-section "E. Hoops," p. 193, is revised to read as follows:

3. *Hoop-Spacing.* The maximum hoop-spacing for either grass or corn silage should be as given in Table 3. This table does not apply for silage with a moisture content above 75 percent, as closer spacing will be required for wetter silage.

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Vol. 17 No. 3 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN January 1946

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Calculation of raw mixes

ARTURO VERA, Jr., *Rock Products*, V. 48, No. 8, p. 109, Aug. 1945. Reviewed by ROY N. YOUNG

The use of determinants facilitates design of 3-component mixes for specific clinker compositions in the manufacture of portland cement. A practical example of the method is given.

Slurry sampling methods

MARK LINTZ, *Rock Products*, V. 48, No. 8, pp. 107-108, Aug. 1945. Reviewed by ROY N. YOUNG

A system for splitting and handling test samples from an automatic sampler is described, involving sufficient agitation to insure uniformity. The use of grab samples from storage tanks gives unreliable results and may cause cyclic errors.

Natural cement blend improves concrete

D. E. LOVEWELL, *Engineering News Record*, V. 135, No. 4 (July 26, 1945) pp. 86-88 Reviewed by S. J. CHAMBERLIN

Improved properties are claimed for concrete made by substituting one bag of air-entraining natural cement for a bag of normal cement in a cubic yard mixture. The mixture is adjusted by decreasing the percentage of sand and the water-content, resulting in comparable strengths and workability. The air-content of the concrete can be accurately controlled by changing the amount of natural cement used.

Large triaxial testing machine built by Bureau of Reclamation

R. F. BLANKS, AND DOUGLAS MCHENRY, *Engineering News Record*, V. 135, No. 6 (Aug. 9, 1945) pp. 113-115 Reviewed by S. J. CHAMBERLIN

Cylindrical specimens, 6 by 12 in., may be tested both axially and laterally under independent pressures as high as 125,000 psi. The walls of the chamber are 15½ in. thick, made of three concentric shrink-fit cylinders of alloy steel with the inner wall prestressed. Specimens are inserted or removed by unscrewing the smaller of two concentric threaded plugs. The ram for axial loading may be removed by unscrewing the plug at the bottom. The machine will be used for determining the shearing strength of concrete and rock, and the behavior of these materials under loadings simulating those obtained in large gravity-type dams.

Form linings for concrete surfaces

HARRISON V. PITTMAN, *Engineering News Record*, V. 135, No. 18 (Nov. 1, 1945), pp. 92-96 Reviewed by S. J. CHAMBERLIN

The lining used at Norfolk Dam was about 0.08 in. thick and consisted of a thin fabric facing adhesively bonded to an absorptive cardboard back. Total cost of the lining was about 12 c. per sq. ft. Backing the absorptive material with a non-absorptive lining, ¼-in. hard-surfaced manufactured board or plywood, produced better surfaces at a savings of 3½ c. per sq. ft. over use directly on wood sheathing, which would have required frequent reconditioning. During cold weather, when the application of water for curing was discontinued, the lining was left in place for 21 days as a curing and protective measure. The lining was effective in minimizing surface pits and voids; eliminating sand streaking; and in producing an even-textured, dense appearing surface.

Paving for the biggest planes in sight

Engineering News Record, V. 135, No. 2 (July 12, 1945), pp. 96-98 Reviewed by S. J. CHAMBERLIN

Designed for a wheel load of 175,000 lb., the slabs are 9, 12 and 18 in. thick, and are supported on a well compacted 24-in. fill. The thickest slab is required on portions of the hangar floor and aprons where there is excessive vibration incidental to warming up the planes. The 20x20-ft. panels of the thick concrete contain only temperature steel and 2-in. dowels on 12-in. centers. Dowels were held in place by split headers and by welded pipe removable jigs. Bayonet-type vibrators were used ahead of the finishing machine. The surfaces were floated, screeded, floated again, belt finished and then broomed. The 12- and 18-in. slabs were cured by ponding to help dissipate the heat of setting.

Portland cement dispersion by adsorption of calcium lignosulfonate

FRED M. ERNSBERGER AND WESLEY G. FRANCE (Ohio State University), *Industrial and Engineering Chemistry*, V. 37, p. 598, June, 1945 AUTHORS' SYNOPSIS

The results of turbidity tests on portland cements suspended in an aqueous medium, with and without addition of calcium lignosulfonate, as obtained with the Wagner turbidimeter, show that the additional agent disperses the suspended cement particles. In the tests reported, the degree of increase in turbidity for two types of portland cement in an aqueous medium is shown to be a function of the amount of adsorbed dispersing agent. It may be concluded that the mechanism of dispersion involves the positive adsorption of lignosulfonate anions by the cement particles. The determination of an adsorption isotherm for calcium lignosulfonate and portland cement suspended in water is described; data are given for a typical cement.

New method of revetting old man river

LT. COL. A. B. PICKETT, *Engineering News Record*, V. 135, No. 12 (Sept. 20, 1945), pp. 124-128 Reviewed by S. J. CHAMBERLIN

The reinforced concrete mats, 24 ft. wide, 60 ft. long, and 1⅛ in. thick are flexible enough to permit their being rolled onto a large timber core at the casting plant. They are then carried by barge to the site, rerolled on a steel drum and unrolled down the river bank under water to final position. Light steel angles were tacked across a tempered wallboard platform to serve as chairs for the wire mesh reinforcement and to serve as spacers for removable metal grid forms. The slotted metal grids, equal in depth to the thickness of the mat, formed V-shaped cuts and served as guides for the screed. After initial set, the grids were lifted and moved ahead to the next platform.

Prefabricated reinforced concrete structures

HERMANN STOESS, *Beton u. Stahlbetonbau*, V. 43, No. 5/6, page 25, Mar. 15, 1944 — Reviewed by ARTHUR V. THEUR

This article describes several systems of precast concrete-unit constructions developed in Germany for barrack or industrial housing purposes. The complete framework as well as the footings, floors, and roof slabs are precast elements. For the filler walls either light weight concrete slabs or burned clay units are used. Details of all structural members as well as methods of jointing are described. The quantities of materials required are stated to be 1.86 pounds of steel and 149 pounds of cement per square yard of floor area, or approximately 3.9 pounds of steel and 37 pounds of cement per cubic yard of covered space.

Temperature effects on mortars

L. R. FORBRICH, *Rock Products*, V. 48, No. 9, pp. 106-7, Aug. 1945. — Reviewed by ROY N. YOUNG

Masonry cement mortar cubes were stored at 70 F., 28 F. or -10 F. during the first 3 days, thereafter in air or water at 70 F. until tested in compression. Strengths of specimens cured in water at 70 F. were 1850, 2400 and 2800 psi at 3, 7 and 28 days. Freezing reduced strength, the maximum reductions at 28 days being 15 per cent for water curing, 53 per cent for air curing. The greatest strength reductions resulted from -10 F. exposure when specimens were fresh. Storage at 70 F. for 6 or 24 hours before freezing lessened the effect of freezing on strength. For cold weather work, mortar having high water retention and relatively rapid hardening is recommended. If freezing occurs, an adequate supply of curing water after thawing is necessary.

Stresses in beams near supports

HERMANN BAY, *Beton u. Stahlbetonbau*, V. 43, No. 7/8 page 40. (April 1944) — Reviewed by ARTHUR V. THEUR

In this discussion two methods of attacking the involved problem of stresses in a beam near supports are presented in some detail. The first of these methods is an analytical study based on the solution of the problem of plane stress distribution in a wedge as derived by Mitchell in England (London Math. Proc. V. 32, p 35, 1900). The second method of solution follows by means of an application of difference equations. No special claims are advanced by the author regarding either method other than that of giving the reader a general indication of the nature of the stress distribution.

The stress distribution as derived by the author is shown by means of graphs for horizontal as well as vertical sections taken at intervals across a short section of beam near a support. The stress trajectories are then plotted and the resulting diagram is compared with similar ones derived by means of photo-elastic studies.

Multiple grouting of a dam foundation

L. F. HARZA, *Engineering News Record*, V. 135, No. 10 (Sept. 6, 1945), pp. 103-105 — Reviewed by S. J. CHAMBERLIN

The first grouting in the basaltic lava foundation of the Rincon del Bonete Dam in Uruguay was carried to about 13 ft. below the concrete cut-off. After 8 or 10 hours the hole was re-drilled to former depth and size and the grouting repeated until the limiting pressure was again reached. The cycle of re-drilling and re-grouting was repeated until negligible additional grout could be injected. The hole was then drilled 13 ft. deeper each time into the rock and the process repeated to a depth of 80 ft. with a rubber "packer" in place to prevent return of the grout above. One group of holes studied were only approximately one-half grouted the first time. The author believes that by several repetitions the various communicating cavities might be filled up in

the order of their resistance of their paths of communication with the grout hole. The author also raises the question of whether the slow rate of grouting due to the equipment and procedure employed accounts for the large re-grouting ratios.

Effect of heat on portland cements containing Vinsol resin

Technical News Bulletin, National Bureau of Standards, No. 340, Aug. 1945 — Highway Research Abstracts

The use of air-entraining cements is rapidly increasing, principally because of their superior resistance to freezing and thawing. One agent commonly used to produce air entrainment is Vinsol resin. Some mills have experienced considerable loss of resin while grinding the flake or powdered material with clinker. This has occurred most frequently when high temperatures (around 300 deg. F.) existed. Leonard Bean and Albert Litvin, of the Bureau, have conducted a study of the effect of heat on Vinsol resin and on its air-entraining properties. Vinsol resin was found to carbonize on heating at 300 F. Heating cement containing the resin for 48 hours at that temperature volatilized as much as two-thirds of the resin and reduced the air entrained by mortars made from the cement. Also, the methoxyl content of Vinsol resin was found to be reduced by heating at 300 F. Heating takes place in the finish grinding of commercial cements, and temperatures of over 250 F. may be found after many days of storage. Methods of determining Vinsol resin are discussed.

Structural behavior of Fontana Dam revealed by dual-purpose instruments

W. R. Wavoh, *Engineering News Record*, V. 135, No. 20 (Nov. 15, 1945), pp. 94-98 — Reviewed by S. J. Chamberlin

A large number of special measuring instruments were located to determine structural behavior as well as data required for construction and data for study of long time behavior. Electrical instruments, including Carlson strain meters and Carlson stress meters, have been embedded in three typical blocks of the dam to determine the stresses and strains near the base of the maximum section and to determine boundary conditions at the upstream and downstream faces. Joint meters were installed across longitudinal and transverse joints to determine changes at contraction joints, to observe movements of adjacent blocks during grouting and to check subsequent changes. Deflection measurements with long plumb lines have proven valuable in indicating the condition of the top grout sections in the longitudinal joints. During construction the embedded electrical instruments which measure temperature were used as the base control for the cooling operation. Measurements of foundation uplift pressures are being compared to assumed pressures as a basis for determining whether additional foundation drains or curtain grouting will be required.

Wintertime concreting at the Soo

Engineering News Record, V. 135, No. 4 (July 26, 1945), pp. 80-85 — Reviewed by S. J. Chamberlin

The wall site was completely enclosed by insulating board, supported on 900 lin. ft. of self-supporting, steel tubular scaffolding about 40 ft. high, to permit concreting operations at temperatures of to −24 F. and snowfall aggregating some 90 in. The timber roof, supported by the scaffolding and the undisturbed bank back of the wall, contained removable panels for installation and removal of the panel forms. Hatches with removable covers were installed to receive the metal hoppers for placing the concrete. Force-blower steam heaters heated the enclosure to a temperature of 50 F. at the bottom and 70 F. under the roof. Steam, water and air lines extended the full length in a sawdust filled timber box conduit. Transit mix concrete made with air-entraining cement was delivered at a minimum temperature of 60 deg. Dumped into buckets, the concrete

was lifted by crane to the hoppers which terminated in elephant trunks for placing. The concrete was cast in alternate monoliths against panel forms lined with an absorptive material consisting of a porous paperboard faced with a ply of light-weight cotton sheeting.

Special steel forms speed concreting at Chicago's water filtration plant

Engineering News Record, V. 134, No. 26 (June 28, 1945), pp. 76-79 Reviewed by S. J. CHAMBERLIN

Special steel forms were used to build the reinforced concrete washwater troughs that had to be level and to within $\frac{1}{32}$ in. of the required elevation. Each trough had a length 25 ft. 9 in. and an inside opening 26 by 17 in. with 3 in. side walls. The forms consisted basically of four side plates for forming the walls and a bottom plate bolted to the sides. The side plate assembly carried a heavy steel walkway on each side and had adjustable ties across the top. Forms were supported on timber frames through screw jacks. Reinforcement was welded into cages approximately equal in length to that of the troughs. Concrete was chuted into catch-boxes from a traveling platform that operated on rollers along the top of the filters and raked into the forms. Concrete first was placed to a 2-in. depth over the bottom and up about three-fourths of the wall height. Excess material that flowed out onto the bottom slab was removed after 30 to 45 min. and the remainder troweled to correct shape and thickness. Additional concrete brought the side walls up to correct elevation. Finished troughs were water-cured and rubbed, if necessary with carborundum stone.

Construction practices in South America

ARTHUR J. BOASE, *Engineering News Record*, V. 135, No. 10 (Sept. 6, 1945), pp. 96-102 Reviewed by S. J. CHAMBERLIN

The author believes that neither the climate nor methods of construction can account for the higher working stresses and the use of more slender members in South American practice. There is some excellent construction and control in individual cases. The concrete is not as strong, strengths for a given water-cement ratio being about 60 per cent of those attained in this country. Control tests of aggregate are seldom made. In general, small, plain bars of structural grade are used. All fabrication is done on the job, mostly by hand labor. Fabricated accessories, such as bar supports, are not commonly used. Form lumber is rougher and produces more and larger fins. Concrete mixtures are generally arbitrary and aggregate is usually measured by volume in wood boxes. Concrete is sometimes distributed by old-fashioned, long, high chutes. Excellent, scientifically controlled concrete was being produced by the modern plant at Volta Redonda, Brazil, for the construction of a steel mill. An interesting feature at the same site was a traveling form-support of reinforced concrete in the shape of an inverted truss.

Scientific sampling for accuracy

HUBERT WOODS, *Rock Products*, V. 48, No. 8, pp. 102, 103, 110, Aug. 1945 Reviewed by ROY N. YOUNG

Sampling of crushed material for process control is frequently conducted in a manner which leads to sampling errors many times greater than the analytical error. The minimum amount of material required for a properly representative sample depends on the maximum particle size of the material and the degree of precision required in the test results. Experimental determination of the minimum quantity is possible, and is well worth while where a material stream is to be sampled periodically. The probable total error is determined by analyses of at least 10 samples cut from a large sample by quartering and riffling, the test sample size being small enough to yield results with

a probable error considerably greater than the desired precision. The probable analytical error is determined by making at least 10 analyses of a single sample. The probable sampling error is computed from the analytical and total error data. The effect of maximum particle size on required sample quantity is determined by repeating the process on portions of the same material crushed to a smaller maximum particle size, a total set consisting of at least 3 samples of different maximum sizes. Formulas are given for computing necessary sample quantities from the probable error data.

Aggregate production at Fontana Dam

JACK B. MCKAMEY, *Engineering News Record*, V. 135, No. 8 (Aug. 23, 1945), pp. 88-94 Reviewed by S. J. CHAMBERLIN

More than six million tons of quartzite was quarried from a restricted area between two heavy slate beds at a site about a mile downstream from the dam. A natural fault divided the quarry so that it was possible to alternate operations at the two ends. As quarrying progressed, a second face was developed 250 ft. above the floor to eliminate hazards and deep drilling. The second face reached a maximum height of 150 ft. Loading proceeded from the two levels. Large boulders were broken by placing a charge of dynamite on top and covering the charge with plastic clay. Rock was loaded by electric shovels into 10 cu. yd. trucks which delivered to 42-in. gyratory crushers. The crushed rock was carried by a 42-in. conveyor belt to primary storage across the river. Coarse aggregate was prepared in two screening and crushing plants which could be operated independently or concurrently. Fine aggregate was produced in one operation from fine stone in rod mills fed from a surge bin. The wet material flowed by gravity to a hydro-separator and rake classifiers. Finished storage of the four sizes of aggregate and sand was in a natural ravine. Steam coils were installed at the sand and fine rock openings of the reclaiming tunnel to prevent freezing and crusting. The material was conveyed back across the river to the concrete plant by belt.

Vacuum processing of Shasta Dam spillway

C. S. RIPPON, *Engineering News Record*, V. 134, No. 24 (June 14, 1945) pp. 93-96 Reviewed by S. J. CHAMBERLIN

The regular form panels for a 5-foot lift were divided into five tiers of five 10-ft. chambers each by plywood strips, sealed to the form with asphaltic compound. The air space was formed by a layer of heavy base wire covered with a layer of fly screen and then a layer of unbleached muslin. A 2-in. pipe extended from the centrally located, reciprocating type compressor and connected by rubber hose to receiving tanks on the back of the forms. A section of hose connected the tanks to a pipe manifold which led off with smaller hose to the vacuum chambers. As soon as enough concrete was compacted against the form to cover one chamber the vacuum was applied. After a short time the concrete near the face was revibrated with the suction maintained. A vacuum of 15 to 20 in. of mercury could be maintained with as many as ten of the chambers operating at one time. An average of 0.58 lb. of water was extracted from each square foot of surface processed at a cost of 20 cents. The indicated compressive strength of the surface was 2.6 times greater, as determined by the "ball impact" test and comparative resistance to sand blast, than that of concrete placed against a wooden form. The vacuum process was also used on the apron floor with movable mats applied to the surface after it had been shaped to line and grade.

Deck bridge built of precast T-beam units

Engineering News Record, V. 135, No. 16 (Oct. 18, 1945), pp. 96-99 Reviewed by S. J. CHAMBERLIN

Part of the 26-ft. clear roadway bridge consists of thirty-three 40-ft. trestle spans supported by 5-pile concrete bents. The 7¼-in. thick slab is carried on five lines of

beams 33 in. deep. Six sections are cast side by side in a row perpendicular to the shoreline in plywood forms which are supported on heavy falsework timbers resting on concrete sills. The forms are stripped rather than lifting the slabs out. Five pairs of bolts are embedded in each end of the slab to permit the section to be raised by a gantry crane for loading on barges. Steel bearing plates are laid flat on the bottom near each end of the beam forms to match 3-in. deep recesses in the caps of the pile bents. The plates contain holes for extending short ¼-in. bolts downward through the forms for the addition of other bearing plates after the slab has been raised. Larger openings in the plates, and cast the depth of the slab, are provided for ¾-in. dowels to be placed from above after the slab has been transferred and aligned over the supporting bents. The recesses in the bent caps are filled with mortar and the sections lowered into position by pumping water into the scows. The load is supported on jacks until the mortar has hardened. The 21-in. square reinforced piles were cast on a 4-in. reinforced concrete slab. Alternate piles were cast first, wooden forms being used for this work only. For the remainder of the piles no forms were used, alternate piles being removed and those left in place used as forms. Caps for the pile bents were cast in place on the driven piles.

Pressure grouting

WILHELM DETIG, *Beton u. Stahlbetonbau*, V. 43, No. 7/8, p. 37. (Apr. 1944)

Reviewed by ARTHUR V. THEUR

The author describes a series of laboratory tests to determine the efficiency of pressure grouting methods. The apparatus used consisted of two steel I-beams 27 ft. long, each with one half of one of its flanges removed so that the webs could be brought together. By means of different thicknesses of spacer plates which could be inserted along the edges between the two beams and a series of closely drilled bolt holes, a closed channel of varying thicknesses could be obtained. Near the end of one of the beams which were laid with webs horizontal, a threaded hole was provided for attaching the grout line. At several locations along the length of the channels, means for measuring pressures were provided.

The experiments were made for three different channel thicknesses, 0 or with the beams bolted together directly, 0.013, and 0.078 inches. Pressures ranged from 280 to 880 psi. For all pressures greater than 590 psi, a peculiar and decided spreading or bulging of the beams occurred.

Using a neat cement made up with 40 per cent by weight of water, complete filling of the 0.078 inch channel was obtained along the full length of 27 feet under a pressure of 590 psi. For a complete filling of the 0.013 inch channel using this same mix, a pressure of 880 psi was required. With a mix consisting of 80 per cent by weight of water, and with the beams bolted directly together, filling of the channel to a distance of 4.1 feet from point of entry was obtained with a pressure of 500 psi. The addition of 1 per cent of plastiment to the several mixes used, increased the distance to which complete filling of the channel could be obtained in every case.

Additional data relating to the adhesiveness of the hardened grout to the beam surfaces and its flexural tensile strength are included.

Two-way prestressed concrete water storage tank

J. ROLAND CARR, *Engineering News-Record*, V. 135, No. 14 (Oct. 4, 1945), pp. 108-113

Reviewed by S. J. CHAMBERLIN

The 4,750,000-gal. tank has an average inside diameter of 156 ft., an average water depth of 33 ft. 3 in. when full and a thin-shell concrete dome rising 24 ft. 9 in. above the walls. The inner wall, 8-in. thick at the top and sloping on the inside to a 21-in. thick-

ness at the bottom, was cast first and the main horizontal bands placed around it. The outer shell was built to a minimum thickness of 4 in. at the top and stepped out at the pilasters to a total thickness of 9¾ in. Both the main wall and the outer shell were placed in 26 vertical sections with movable forms. The vertical prestressed rods of $\frac{43}{44}$-in. diam. were placed 4 in. inside of the main hoops at 12-in. centers and 1 in. outside of the hoops at 24-in. centers. The vertical rods were greased, spirally wrapped with building paper and hung from outriggers spiked to the top of the timber forms. The nut and 2½-in. washer at the top of each rod were countersunk below the top of the wall in a recess formed by attaching a waxed paper cup filled with plaster. After the concrete had been cast and cured the plaster was removed and the inner rods were stressed to 32,000 psi. (15000 psi in the outer row) by tightening down the nuts to a predetermined amount as measured by an ordinary micrometer depth gage. Theoretically the vertical prestress in the concrete varied from 120 psi on the thinnest section of the wall to 45 psi at the base. Each of the 81 horizontal hoops was made of 13 rods joined together with turnbuckles. Rods were not upset at their ends but had cold rolled threads to develop their tensile strength. The hoops were stressed to 32,000 psi, as determined by a 4-ft. job-made extensometer, by adjusting the turnbuckles. At the end of a 20-day period in which some of the stress had been lost the hoops were retightened to the design stress. Hoops were adjusted in banks of eight, the rods being prevented from twisting by welding short vertical pieces of ½-in. reinforcing rods to them. Some 80,000 b. ft. of timber was required for the inside dome form. A beveled hinged joint was formed between the parapet and dome to permit rotation.

Complete buildings constructed with precast reinforced concrete elements

B. Loesser, *Beton u. Stahlbetonbau*, V. 43, No. 9/10, 11/12, and 13/14
May, June and July, 1944 Reviewed by Arthur V. Theur

In this series of three articles Professor Loesser describes recent German activity in the field of prefabricated reinforced concrete-unit building construction. While recognizing the considerable number of buildings of this type constructed prior to 1940, little he states, was accomplished in the way of standardization of the units, their dimensions, shapes and detailing. It is in this respect that uniqueness is claimed for the several structures under discussion. For much of the pioneering work in standardization credit is assigned to the I. G. Farbenindustrie whose mill-type of building is given a first rating.

After a discussion of the advantages and disadvantages of the precast unit as against the monolithic structure, the author presents a detailed analysis of the design of footings for the precast job. For the type of footing adopted the column is inserted into a hole provided for the purpose. This hole is slightly larger than the column cross-section and the column is supported on chairs inserted between the footing and column base. After the column has been brought up to proper elevation it is wedged into alignment and the open space around and underneath the column is filled with grout.

Eight different cross-sectional shapes of beams were standardized and nine different types of roof construction developed. The type of roof construction used depends largely on the length and number of spans to be covered, and includes all of those types, —simple beam, tied arch, truss, saw tooth, and cantilevered beam with monitor,— commonly found in conventional construction. The author touches briefly the subject of permissible stresses. He also describes the details of various interlocking joints secured by means of dowels and recesses, and devotes considerable space to the subject of erection equipment.

In the third article of the series the author includes a large number of photographs illustrating a number of structures in different stages of erection.

Brazilian concrete building design compared with United States practice

ARTHUR J. BOASE, *Engineering News Record*, V. 134, No. 26 (June 28, 1945), pp. 80-88 Reviewed by S. J. CHAMBERLIN

The author makes a quantitative comparative study of the Brazilian code and the ACI Building regulations by redesigning a 16-story apartment building recently erected in Rio de Janeiro. As built, the 3.15-in. slab is reinforced with $\frac{3}{16}$-in. plain rounds, interior beams over the partitions are 3.15 or 3.93 in. wide and 22.8 in. deep, and front and rear girders are 8.2 in. wide. Redesigned, the slab would be 4 in. with $\frac{3}{8}$-in. rounds, the interior beams would be 6 by 24 in. and the spandrel beams would be 8 by 32 in. in front and rear walls and 8 by 24 in. in side walls. Brazilian column design is illustrated in two design charts for tied columns with 2500-psi concrete. Data are incorporated from the ACI code for comparison. Because of the lower strength cement used, the Brazilian code does not recognize concrete strengths greater than 2,670 psi for building construction. Material costs are higher and labor costs are lower than in the United States. In the relative cost studies the Brazilian costs were computed at 35 c. per cu. ft. for concrete, 15 c. per sq. ft. for forms and 11½ c. per lb. for reinforcement.

The article summarizes the principal points of difference in the two codes:

"American practice, using intermediate grade steel, would require 26 per cent more reinforcing steel in the floor system than does the Brazilian design, where structural grade was used. Under these conditions the American design uses slightly less reinforcing steel per cubic yard of concrete than does the Brazilian.

"Using the same strength of concrete, the American design requires 32 per cent more concrete in the floor system than does the Brazilian design.

"For axial load on columns the Brazilian code allows 85 per cent higher stress on concrete and 66 per cent higher stress on steel.

"The American tied column has about twice as much concrete, and one and one-half times as much steel as does the Brazilian tied column. A comparison when bending in the columns is considered as still more in favor of the Brazilian type.

"Size of columns will change less frequently under the Brazilian code than it will under American practice, thus permitting large savings in formwork.

"American columns are approximately 60 per cent more costly than are the Brazilian columns for 2,500-lb. concrete. American tied columns of 2,500-lb. concrete have approximately 80 per cent more volume than have the Brazilian columns."

Rigid type pavement and joint spacing

H. F. Clemmer (in reply to the discussion by Fred Grumm, Highway Research Abstracts, Oct. 1944 HIGHWAY RESEARCH ABSTRACTS

It is satisfying to know of the general agreement of the California Division of Highways with the data presented. It is of interest to report that data obtained during the year since this report was presented and the performance of the pavements referred to are in further agreement with the conclusions of the paper.

Mr. Grumm states their experiences has not indicated the need for cutting dummy groove joints to one-half the depth of the concrete slab as reported by this Department. It is stated in the paper that "It is desirable to divide the concrete, only enough to cause cracking, so as to obtain maximum shear value from the irregular edges and to provide maximum area in contact should the pavement be subjected to a high compressive stress". Further discussion explained that this Department had found it necessary to cut the groove one-half the depth of the slab to be sure of forming the planes of weakness. Where the design depends upon the close spacing of joints it is most improtant

to be sure of producing these planes of weakness and it was desired to caution against assuming the planes of weakness were properly produced. It is preferable to sacrifice a small degree of shear value rather than to fail to produce the plane of weakness. A careful check should be made of the completed pavement slab to determine if separation has occurred at the designated locations.

Longitudinal planes of weakness are produced with less difficulty because warping in this direction is not restrained so much as it is transversely; however failure to cut a deep enough groove to create a center longitudinal joint in one State project, constructed during this emergency, resulted in the cracking of the slab some distance from the groove and at a point directly under traffic loads.

Mr. Grumm suggested that load transfer installations may have had an effect in prohibiting the formation of the planes of weakness as reported in this paper. The particular study reported upon as to the depth of the groove required to create planes of weakness, was of plain concrete, that is without reinforcing or the load transfer devices. Undoubtedly, there are many conditions affecting requirements for producing dummy groove joints — such as richness of mix, temperature at time of placing concrete, rate of setting of the cement, curing conditions, subgrade friction etc.

This Department is not quite in agreement with Mr. Grumm's statement that because of the apparent success of the construction of pavement slabs without mesh or bar reinforcement or the use of load transfer units that the use of these materials has no particular value. As discussed in the paper it is believed that reinforcement is of particular value in distributing shrinkage stresses, developed during hydration of the cement, which tends towards the formation of incipient cracks that may later develop into structural failures. Concrete pavements constructed during the last ten years in the District of Columbia show practically no structural failures, that is no cracks or corner breaks, and it is believed the use of mesh reinforcement and load transfer units has been an important factor in this result.

The use of reinforcement and load transfer units is evidently considered excellent insurance by many engineers as indicated by the number of state highway departments that have adopted this design.

With this great improvement in general design, engineers responsible for the planning and construction of the enormous postwar highway program, should be far more concerned than before as to whether or not a change in design of pavement will increase the length of service of the pavement. It is believed that too often design features are based on satisfactory service for five or ten years rather than on careful study of the ultimate results that may be obtained. It is believed that the added cost of including reinforcement and the use of load transfer units in the construction of pavements is splendid insurance and their use will prove to be a factor in extending the service of a concrete pavement slab to the maximum.

ACI NEWS LETTER

Vol. 17 No. 3 JOURNAL of the AMERICAN CONCRETE INSTITUTE January 1946

42nd ANNUAL ACI CONVENTION
Buffalo—Feb. 18-21, 1946

The program for the Institute's Forty-second Annual Convention to be held Hotel Statler, Buffalo, N. Y., February 19, 20 and 21, has been developed to the point of announcing with a good deal of confidence a promising array of papers on many timely and important subjects. Not all of this material can at this writing be definitely announced; in some instances papers now in the making, are still to be selected. The Publications Committee under the Chairmanship of Robert F. Blanks, with the responsibilities of the convention program, has worked under unusual difficulties—difficulties with the elements of time and indecision. Much work has been done in a short period; the program has not completely "jelled". What is reasonably assured, however, will in itself make a good convention—what is just in the offing, and not at the stage of announcement, is due to strengthen the program by selection from a number of outstanding items.

Scheduled a year ago for Hotel New Yorker, New York City, based on the possibility of a full-fledged convention by the removal of war-time control of meetings, arrangements were confirmed by the Board of Direction in September last. Late in November, a meeting was called in New York City of a few nembers of the Institute in that territory who might be of assistance in connection with the convention, with every expectation of an outstanding meeting in that city. Almost immediately after these arrangements, the problems of transportation and of hotel accommodations raised serious questions, and after careful consideration of the whole situation, the Executive Committee of the Board made a change. We were graciously released from our arrangements at Hotel New Yorker and welcomed to Hotel Statler in Buffalo, with strong assurances of the ability of the Statler to take care of all ACI comers.

With the ballots, mailed to all ACI members, Dec. 19 last, went a corrected announcement with reference to convention plans. It now appears that the transportation situation reached a high point of disorder and uncertainty at the holiday season. As to reservations of accommodations at the Hotel Statler, see p. 9. Perhaps it is not necessary to emphasize the importance of making reservation of transportation accommodations at the earliest possible date under existing regulations.

As these *News Letter* pages (the last to go to the printer for the January JOURNAL) are "made up," the situation is as follows:

Board and Committee Meetings

Meetings of the Advisory Committee and of the Board of Direction are scheduled for Monday February 18.

Committee 318, Standard Building Code, A. J. Boase, Chairman, Roy R. Zipprodt, Secretary, is scheduled to meet for a preliminary session, 9:30 a. m. Tuesday February 19, with the expectation of continuing its labors the afternoon of the 21st following the adjournment of the convention and possibly through Friday, the 22nd.

Committee 711, Precast Floor Systems for Houses, F. N. Menefee, Chairman, is scheduled to meet the morning of February 19 at 10:30 for possible last minute action in reference to the presentation of the Committee's report (published in the *Proceedings* pages of this JOURNAL) for adoption as an Institute Standard.

John R. Nichols, chairman Sub-committee 5, of Committee 318, has called a meeting of that sub-committee at a time and place to be announced on the convention bulletin board.

Further announcements will be made of technical committee meetings at convention time as their chairmen decide and notify us.

CONVENTION SESSIONS

General convention registration begins at 9:30 a.m., February 19.

The first general session of the convention will open at 2 o'clock, Tuesday, February 19, and will cover a variety of subject matter. (See note of that variety, p. 4).

Air Entrainment in Concrete

Tuesday evening, February 19, will be devoted to a consideration of the development of rapidly increasing experience with air entrainment in concrete. Frank H. Jackson will lead that session.

The success of the 1944 convention session on air entraining concrete has prompted the schedule of another session on this most inter-

esting and timely subject. The same general procedure will be followed, that is, there will be one major paper covering certain broad aspects of the problem, followed by a series of short 10-minute discussions on particular phases. These will be presented by a selected group of participants, all of whom are familiar with the latest developments in the particular fields which they will cover. Subjects include the use of air-entraining concrete in ready-mix concrete operations, in the manufacture of concrete products, in highway and airport runway construction, etc. The comparative merits of various methods of securing air entrainment—by the use of air-entraining cements, air-entraining agents added at the mixer, and air-entraining natural cements blended with portland cement will be discussed, as well as various methods of field control, including recently developed procedures for determining air content of concrete by direct volumetric measurement. Opportunity for general discussion will be provided insofar as possible.

Research

At 9:30 a.m. Wednesday, February 20, the session will be under the auspices of ACI Committee 115, Research, Morton O. Withey, Chairman, S. J. Chamberlin, Secretary. Every year Professor Chamberlin queries the membership of the large Research Committee and other sources of information. From the reports made some research projects are listed for outline presentation. The annual sessions of this committee have increased in interest and attendance throughout the years.

Repair and Maintenance of Concrete

Wednesday afternoon, February 20, there will be a session under the guidance of Roderick B. Young, past president of the Institute, Chairman ACI Committee Department 800, Repair and Maintenance, as the basis for committee studies in Department 800. Mr. Young, who will lead off with a contribution of his own, has been successful in developing a series of papers on repair and maintenance of concrete in a variety of work. One will discuss "Hydraulic Structure Maintenance in Using Pneumatically Placed Mortar"; another, "Methods of Repair of Two 175-ft. Concrete Chimneys with Minimum Shutdown"; others will consider "Maintenance and Repair of Concrete Pavements"; "Maintenance and Repair of Concrete Bridges on the Oregon Highway System" by G. S. Paxson. (See Nov. 1945 JOURNAL.) Still another paper, contrasting workmanship on two hydraulic structures, will report the repair procedure on one

of them; and there is one in the series which describes some of the methods used in repair, restoration and stabilization of structures in need of repair, or strengthening to meet increased use and loads.

Election Returns—Presentation of Honor Awards—R/C Design

At 8 o'clock Wednesday evening, February 20, a general session will open with a report on the annual election followed by the address of President Parsons, and presentation of awards, and will continue with the development of interesting themes in reinforced concrete design theory and practice under the leadership of A. J. Boase.

A paper by Douglas McHenry will describe a design method for determining internal stresses in concrete structures by means of a lattice analogy which is solved by successive approximations involving only simple arithmetic. The theory of the method was published in a paper by McHenry in the Journal of the Institution of Civil Engineers (London). The present paper will be concerned chiefly with examples of the application of the method.

Construction Practice—Emphasis on War

The final convention session, Thursday morning, will take up examples of construction practice in the field of concrete with some emphasis on war experience, under the leadership of Lewis H. Tuthill.

A Variety of Subject Matter Not Yet Placed on Schedule

"Membrane Waterproofing" will be the subject of a paper to come from Bureau of Reclamation studies.

John R. Nichols will discuss the matter of "radiant heat" in its relation to reinforced concrete floors, walls, etc.

New design ideas for precast, reinforced concrete members "welded" with pneumatically placed mortar will be presented and there will be a report of their war-time application to a ship.

Also: A report on atomic bomb effects on concrete in Japan; use of coral aggregates at Pacific bases, and other important items.

Committee 711, F. N. Menefee, Chairman, will present its report on "Proposed Minimum Standard Requirements for Precast Concrete Floor Units," on a motion for adoption as an ACI standard.

Committee 714, William W. Gurney, Chairman, will present "Proposed Recommended Practice for the Construction of Concrete Farm Silos" for adoption as an ACI Standard.

WHO'S WHO in this JOURNAL

Gerald K. Pickett

(see p. 165) a comparatively new ACI Member, is nevertheless not new to these pages. Since 1940 he has been identified with the basic research division of the Portland Cement Association Research Laboratory and only recently (September, 1945) returned to the faculty of Kansas State College, Manhattan, Kan., where he is Professor of Applied Mechanics.

Stanley U. Benscoter

structural engineer with the Navy Department's Bureau of Aeronautics, Washington, though not yet an ACI Member, is not unknown to ACI readers. To the February 1943 Journal he contributed "A Semi-Circular Arched Conduit with Uniform Symmetrical Loading." (See also *News Letter* p. 6 of that issue). His present contribution, "Floating Block Theory in Structural Analysis" appears p. 205, this Journal.

Howard R. Staley and Dean Peabody, Jr.

contribute "Shrinkage and Plastic Flow of Prestressed Concrete" to this issue (p. 229).

Howard R. Staley, ACI Member since 1937, is Assistant Professor of Building Construction in the Department of Building Engineering and Construction at the M.I.T. He was born in Iowa nad received the degree of S.B. in 1935 and S.M. in 1937 from M.I.T. He is a member of Tau Beta Pi, Chi Epsilon and Sigma Xi. He has been a member of the instructing staff of M.I.T. since 1937. For the twelve years previous to 1932 he was a member of the building firm of B. S. Staley and Sons, Centerville, Iowa. He has presented papers to the Designers Section of the Boston Society of Civil Engineers and to the American Society for Testing Materials, of both which organizations he is a member. He is a member also of A.S.C.E., S.P.E.E., A.A.A.S.

Dean Peabody, Jr., ACI Member since 1914, and member of Committee 318, Standard Building Code, is Associate Professor of Structural Design in the Department of Building Engineering and Construction at the Massachusetts Institute of Technology. He was born in Massachusetts and received his S. B. degree from M.I.T. in 1910 and the S.M. degree in 1938. He is a member of Chi Epsilon and Sigma Xi. He has been a member of the instructing staff of M.I.T. since 1910, including one year as exchange instructor at the Worcester Polytechnic Institute. During the summers of 1911 to 1916 he worked in the field or the design offices of the Aberthaw Construction Co.; New England Concrete Construction Co.; Lockwood, Greene and Co.; and Stone and Webster. He is author of "Reinforced Concrete Structures" and has presented several papers to the Designer's Section of the Boston Society of Civil Engineers of which he is a member. He is also a member of A.S.C.E.; A.S.T.M.; A.A.A.S.

For further convention subject matter and a convention time-table see February Journal — or, about February 1 ask the ACI secretary for an advance program.

New Members

The Board of Direction approved 47 applications for Membership (42 Individual, 2 Corporation, 1 Junior, 2 Student) received in October and November as follows:

Abel, G. C., 23 View Road, London N. 6, England

Allen, Herbert C., 303 Geneva St., Highland Park 3, Mich.

Anjel, Anton A., Box 112, Cocoli, Canal Zone

Booth, John J., 117 Dilworth Bldgs., Cor. Queen & Customs Sts., Auckland, New Zealand

Brooks, Samuel H., 326 Center St., East Pittsburgh, Pa.

Cafaro, Martin, 309 East 28th St., New York 16, N. Y.

Cameron, L. S., 3428 Ettie St., Oakland 8, Calif.

Campbell, W. E., 319 S. 12th St., New Castle, Ind.

Cardenas, Victor Morales y de, Compostela 158, Havana, Cuba

Chang, Wan-chiu, Engineering Dept., Central China, Railways Administration North Station, Shanghai, China

Chen, Manning, c/o Mr. R. A. Kane, Room 1010 U. P. Headquarters Bldg., 1416 Dodge St., Omaha 2, Nebr.

Crowe, George F., 2801 Colonial Ave., Norfolk 8, Va.

Daymude, Charles A., Dept. of Bldgs. & Safety Engineering, 555 Clinton St., Detroit 26, Mich.

Dayton, Cedric L., 16 Hollywood Ave., Tuckahoe 7, N. Y.

Ernst, E. S., Dewey Portland Cement Co., Box 749, Davenport, Iowa

Flores, Rodrigo, Ramon Nieto 920 of 306, Santiago, Chile

Fox, Carl, 887 W. Division St., Decatur, Ill.

Garing, Athol C., 606 Aspen, Coulee Dam, Wash.

Gundmundsson, Jon Sig., P. O. Box 396, Reykjavk, Iceland

Haggerty, Francis V. J., c/o Warner Co., 200 S. Market St., Wilmington, Del.

Harrison, Marc M., 45 Popham Road, Scarsdale, N. Y.

Holbrook, Albert E., 129 Ivanhoe St., S. W., Washington 20, D. C.

Huerta, Ing. Manuel Castro, Monte Alban No. 39, Col. Narvarte, Mexico

Jeffers, Paul E., 810 West Fifth St., Los Angeles 13, Calif.

Knaust, Adam, Marlboro, N. Y.

Kosseim, Emile, 2F Rue Said, Heliopolis, Egypt

Laboratory of Strength of Materials, Beaucheff 850, Santiago, Chile (Sr. Edmundo Thomas)

Ledon, Rolando Castaneda, Edificio America, Jovellar y N, Dpto 210, Havana, Cuba

Leduc, Joseph R., Ecole Polytechnique, 1430 St. Denis St., Montreal 18, Que., Canada

Lewis, Elbert F., Box 573, Greensboro, N. C.

Lundteigen, A. Jr., Ash Grove Lime & Portland Cement Co., Louisville, Nebr.

McKean, J. L., 17 Kelvinside Terrace, Glasgow N. W., Scotland

Mercer Steel Company Inc., 838 N. W. 13th Ave., Portland 9, Ore.

Miles, M. A., c/o Ayres, Lewis, Norris & May, 506 Wolverine Bldg., Ann Arbor, Mich.

Oliva, Francisco Santos, Tepeji No. 85, Mexico D. F., Mexico

Paredes-Manrique, Luis, Cia de Petroleo Shell Bogota, Colombia, S. A.

Porter, George W., 6729 Sycamore, Ave. Seattle 7, Wash.

Rathod, M. P. B. E. (Civil), Jiva Devsi Bldg. No. 2 Room No. 19, Ranade Road, Dadar (B. B.) Bombay 14, India

Sara, Allan M., 29 Church St., Parramatta, N. S. W., Australia

Spencer, Ernest L., Northeastern University, 360 Huntington Ave., Boston, Mass.

Staples, Leroy A., 423—15th St., Alexandria, La.

Suarez, Angel Cano, F y 2 La Sierra, Havana, Cuba

Tajirian, Faraj, M. I. T. Dormitories, Box 60, Cambridge 39, Mass.

Tobalina, Bernardo Vazquez y, Luz Caballero Oeste 464 altos, J. del Monte, Havana, Cuba

Tolley, H. F., U. S. Bureau of Reclamation, Klamath Falls, Ore.

Valdez, Alberto Andrue, 10 de Octubre 1369, J. del Monte, Havana, Cuba

Winner, Walter I., 303 Ferry, Coulee Dam, Wash.

Honor Roll

February 1 to December 31, 1945

Rene Pulido y Morales, in Havana, Cuba heads the list with 24 new Members proposed since Feb. 1. The current Honor Roll closes with applications received by January 31, 1946.

Rene Pulido y Morales **24**
Roy Zipprodt **7½**
H. F. Gonnerman **6**
Harry B. Dickens **5**
A. Amirikian **3**
J. A. Croft **3**
Ernst Gruenwald **3**
Dean Peabody, Jr. **3**
J. H. Spilkin **3**
Charles S. Whitney **3**
Francis MacLeay **2½**
D. E. Parsons **2½**
Charles E. Wuerpel **2½**
C. Blaschitz **2**
H. W. Cormack **2**
J. W. Kelly **2**
Calvin C. Oleson **2**
O. G. Patch **2**
C. H. Scholer **2**
J. M. Wells **2**
J. C. Witt **2**

Ben E. Nutter 1½
O. A. Aisher 1
J. B. Alexander 1
Michel Bakhoun 1
H. C. Bruce 1
H. Victor Carman 1
W. Fisher Cassie 1
A. R. Collins 1
R. F. Dierking 1
H. F. Faulkner 1
P. J. Freeman 1
B. F. Friberg 1
J. K. Gannett 1
Arturo Gantes 1
Stanley S. Haendel 1
W. S. Hanna 1
G. H. Hodgson 1
F. B. Hornibrook 1
V. P. Jensen 1
L. I. Johnstone 1
William G. McFarland 1
Denis Matthews 1
Charles E. Morgan 1
H. W. Mundt 1
Y. G. Patel 1
J. R. Pattilo 1
A. F. Penny 1
Kenneth Powers 1
Guy Richards 1
A. T. Rogers 1
Simeon Ross 1
John A. Ruhling 1
Herman Schorer 1
Byram Steel 1
G. W. Stokes 1
H. D. Sullivan 1
Wm. Summers Jr. 1
M. A. Timlin 1
J. W. Tinkler 1
Lewis H. Tuthill 1
Maxwell Upson 1
Stanton Walker 1

The following credits are, in each instance, "50-50" with another Member.

Birger Arneberg	P. M. Ferguson
E. E. Bauer	Alexander Foster
E. W. Bauman	G. L. Freeman
P. G. Bowie	Grayson Gill
C. H. Chubb	E. A. Gramstorff
J. H. Chubb	N. M. Hadley
Arthur P. Clark	W. C. Hanna
R. R. Coghlan	Carl W. Hunt
R. B. Crepps	A. Dovali Jaime
R. A. Crysler	W. R. Johnson
Harmer E. Davis	Paul A. Jones
J. L. Drucke	H. J. McGillivray

R. E. McLaughlin
Adolph Meyer
A. F. Moore
O. F. Moore
E. Nennigar
M. D. Olver
C. E. O'Rourke
Lucien Perrault
Milos Polivka
Jerome Raphael
F. E. Richart
Chas. S. Rippon
D. O. Robinson
Kanwar Sain
J. L. Savage
Oskar Schreier
A. L. Strong
A. C. Trice
K. Tsutsumi
J. H. de W. Waller
S. J. Warberg
A. R. Waters
David Watstein
H. J. Whitten
George Winter
Harry C. Witter
S. H. Woodard
K. B. Woods
L. Zeevaert

John J. Earley

John Joseph Earley, past president (1938) American Concrete Institute, Henry C. Turner Gold Medalist (1934) for "outstanding achievement in developing concrete as an architectural medium" and one of the Institute's 15 members elected to Honorary membership, died November 25, 1945 at his home 2701 Connecticut Ave. N. W., Washington, D. C., after a long illness.

Mr. Earley was an architectural sculptor fifth in a line of such artists, himself a native of New York. He was an apprentice in his father's studio, specializing chiefly in ecclesiastical sculpture, at the age of 17. His death occurred at the age of 64.

Entering the contracting business, specializing largely in plaster and stucco, 1906, his work led him to a study of the possibilities of architectural concrete of the exposed aggregate type, and from that, strongly influenced by the lavish color of Byzantine architecture, he made an exhaustive study of the possibilities of what he called plastic mosaic, which at far less cost, would achieve some of the effects of the early mosaics with their employment of high talents of art and workmanship. One of the early records of this development is to be found in Washington's Church of the Sacred Heart, one of a score of United States churches to which he gave his decorative talent—the Baha'i Temple, Evanston, Ill., with its white quartz concrete tracery, perhaps his greatest work.

To the Earley Studio, Rosslyn, Va., he brought as materials for his artist "palette", crushed aggregates selected for durability and color—marble, glass, ceramics, arranged by the Munsell system of color notation. He used to say: "Imagine the day before there was any system of recording musical notes, and you have the present condition so far as general practice goes, in the notation of color".

John J. Earley

He gained fame in converting the Lorado Taft "Fountain of Time," Chicago, into relatively durable form, when all the usual media of the sculptor had to be passed up because of the enormous cost involved for the symbolic group more than 100 ft. long. Mr. Earley's paper "Building the Fountain of Time," ACI Proceedings Vol. 19, won the Wason medal for "the most meritorious paper" of the year.

A sketchy, outline record of his contributions to the technology of concrete and its application to architectural works, will be found in naming some of Mr. Earley's papers published by the Institute:

"Some Problems in Devising A New Finish for Concrete," V. 14; "New

Developments for Surface Treated Concrete and Stucco" (as co-author with J. C. Pearson) V. 16. Then came "Building the Fountain of Time", with its description of the making and use of a 3000-piece mold for the placing of concrete using crushed Potomac River gravel as the predominating aggregate; two papers, each under the title "Architectural Concrete" V. 20 and 22: "Time as a Factor in Making Portland Cement Stucco", V. 23; "The Project of Ornamenting the Baha'i Temple Dome" V. 29; "Architectural Concrete of the Exposed Aggregate Type", V. 30; "Architectural Concrete Makes Prefabricated Houses Possible" V. 31; "Mosaic Ceilings, U. S. Department of Justice Building" V. 31; "On the Work of the Committee on Architectural Concrete of the Exposed Aggregate Type and the Thomas Alva Edison Memorial Tower," V. 34; "The Characteristics of Concrete for Architectural Use," V. 35.

These contributions of Mr. Earley to the work of the Institute, and to the field of concrete, were unique. Unique though his contribution was, it brought sharply into focus the necessity for concrete of very high quality for all exposed ornament. He had haunted the Bureau of Standards, and other sources of data, to find the path that would lead him to mixes best suited to his special purposes. At the Bureau he found a typical plotting of a large number of tests of quality concrete, with the characteristic curve, following close to the medial line among the dots of the individual test specimens. In these he was not interested, but he found one dot barely inside the top margin of the diagram showing a concrete specimen of unusually high quality, and he could not be satisfied until he had found the record of what that concrete was—what made it so good, neglected as it was in plotting the curve.

Always dominated by the artistic approach and with a keen sense of drama, he could bring himself and endless patience to the study of the engineering approach to the entegrity of a material. With that he was also a salesman of a super type. Because he maintained that writing was slow and difficult for him his papers are therefore interesting as specimens of English composition. He insisted that to set down a principle is sufficient, and only rarely did he record the details of means to apply the principle to his particular work. As a young man he was no mean athlete—he played handball with the best, was Washington's champion fencer (crossed points with the late President Theodore Roosevelt), and was a strong swimmer.

He was a member of the Board of Direction as Director Fourth District, 1924 and 1925 and again 1932 and 1934; he was elected Vice-President 1926 and resigned in shying from election to the Presidency. Again he was Vice President, 1936 and 1937, and was elected President in 1938; served as past president member of the Board from 1939 to 1943.

Convention Hotel Reservations

Following is an announcement of rates for room accommodations at Hotel Statler, Buffalo, effective for those attending the convention of the American Concrete Institute February 18 to 21. Make your reservations at once. It is well to ask for a confirmation; it is important to identify yourself with the American Concrete Institute. Address: Mr. Theodore Krueger, Manager, Hotel Statler, Buffalo 5, New York.

Unless requested otherwise, the hotel will hold your reservation until 9 p.m. of the day of your arrival. Give date arriving, and hour.

Room and Bath for One Per Day	3.30 3.85 4.40	4.95 5.50	6.05 6.60
Double-Bed Room with Bath For Two—Per Day	5.50 6.05 6.60	7.15 7.70	8.25 8.80
Twin-Bed Room with Bath For Two—Per Day	6.60 7.70 8.25	8.80 9.35	9.90 11.00
Three Persons in One Room	7.70	8.80	9.90
SUITE—Living Room, Bed Room and Bath For One—Per Day For Two—Double Bed —Twin Beds		 16.00	 12.50 14.70 19.00

If a room at the rate requested is unavailable, one at the nearest available rate will be reserved.

Sources of Equipment, Materials, and Services

A reference list of advertisers who participated in the Fourth Annual Technical Progress Issue of the ACI JOURNAL—the pages indicated will be found in the February 1945 issue and (when it is completed) in V. 41, ACI Proceedings. **Watch for the 5th Annual Technical Progress Section in the February 1946 JOURNAL.**

Concrete Products Plant Equipment

page

Besser Manufacturing Co., 800 45th St., Alpena, Mich. 428
—Concrete products plant equipment, production

Stearns Manufacturing Co., Inc., Adrian, Mich. 387
—Vibration and tamp type block machines, mixers and skip loaders

Construction Equipment

Baily Vibrator Co., 1526 Wood St., Philadelphia 2, Pa. 407
—Concrete vibrators

Blaw-Knox Division of Blaw-Knox Co., Farmers Bank Bldg., Pittsburgh, Pa. 394-5
—Truck mixer loading and bulk cement plants, road building equipment, buckets, batching plants, steel forms

Butler Bin Co., Waukesha, Wis. 421
—Central mix, ready-mix, bulk cement and batching plants, cement handling equipment

Chain Belt Co. of Milwaukee, Milwaukee, Wis. 418-9
—Mixers, pavers, pumps

Electric Tamper & Equipment Co., Ludington, Mich. 410-11
—Concrete vibrators

Flexible Road Joint Machine Co., Warren, Ohio 389
—Pavement joint and joint installers

Fuller Co., Catasauqua, Pa. 377
—Unloading and conveying pulverized materials

Heltzel Steel Form & Iron Co., Warren, Ohio 378-9
—Pavement expansion joint beams

Jaeger Machine Co., The, Columbus, Ohio 392-3
—Concrete paving equipment

C. S. Johnson Co., The, Champaign, Ill. 429
—Mixing and batching plants, buckets, elevators

Kelley Electric Machine Co., 287 Hinman Ave., Buffalo 17, N. Y. 422-3
—Floor finishing equipment

Koehring Co., Milwaukee, Wis. 380
—Tilting and non-tilting construction mixers

Mall Tool Co., 7703 So. Chicago Ave., Chicago 19, Ill. 427
—Concrete vibrators

Master Vibrator Co., Dayton 1, Ohio 430-1
—Concrete vibrators

Ransome Machinery Co., Dunellen, N. J. 433
—Mixers—paving, truck

Viber Co., 726 So. Flower St., Burbank, Calif. 381
—Concrete vibrators

Contractors, Engineers and Special Services

Kalman Floor Co., Inc., 110 E. 42nd St., New York 17, N. Y. 390-1
—Floor finishing methods

Prepakt Concrete Co., The, and Intrusion-Prepakt, Inc., Union Commerce Bldg., Cleveland 14, Ohio 397-400
—Pressure filled concrete

Raymond Concrete Pile Co., 140 Cedar St., New York 6, N. Y. 396
—Pile foundations

Roberts and Schaefer Co., 307 No. Michigan Ave., Chicago 1, Ill. 426
—Thin shell concrete roofs

Scientific Concrete Service Corp., McLachlen Bldg., Washington, D. C. 416
—Mix controls and records

Vacuum Concrete, Inc., 4210 Sansom St., Philadelphia 4, Pa. 409
—Suction control of water in the concrete

Materials and Accessories

Calcium Chloride Assn., The, 4145 Penobscot Bldg., Detroit 26, Mich. 420
—Calcium chloride

Concrete Masonry Products Co., 140 W. 65th St., Chicago, Ill. 385
—Non-shrink metallic aggregate

Dewey and Almy Chemical Co., Cambridge, Mass. 412-5
—Air-entraining and plasticising agents

Horn Co., A. C., Long Island City 1, N. Y. 417
—Waterproofing

Hunt Process Co., 7012 Stanford Ave., Los Angeles 1, Calif. 425
—Curing compound

Inland Steel Co., The, 38 So. Dearborn St., Chicago 3, Ill. 384
—Reinforcing bars

Lone Star Cement Corp., 342 Madison Ave., N. Y. 382-3
—Portland cements

Master Builders Co., The, Cleveland, Ohio, Toronto, Ont. 402-5
—Cement dispersing and air-entraining agents

Rail Steel Bar Association 386
—Reinforcement bars

Richmond Screw Anchor Co., Inc., 816 Liberty Ave., Brooklyn 8, N. Y. 408
—Form tying devices

Sika Chemical Corp., 37 Gregory Ave., Passaic, N. J. 434-5
—Waterproofings, plasticizer, and densifier

United States Rubber Co., Rockefeller Center, New York 20, N. Y. 401
—Form lining

Testing Equipment

American Machine & Metals, Inc., East Moline, Ill. 388
—Riehle hydraulic testing machines

ACI publications in large current demand

ACI Standards—1945

148 pages, 6x9 reprinting ACI current standards: Building Regulations for Reinforced Concrete (ACI 318-41); three recommended practices: Use of Metal Supports for Reinforcement (ACI-319-42); Measuring, Mixing and Placing Concrete (ACI 614-42); Design of Concrete Mixes (ACI 613-44); and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—all between two covers, $1.50 per copy—to ACI Members, $1.00.

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer," and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944. $1.25 per copy; 75 cents to Members.

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940; 140 pages. Price $1.50—to ACI members $1.00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

Concrete Primer (Feb. 1928)

Prepared for ACI by F. R. McMillan, it had five separate printings by the Institute alone (totalling nearly 70,000 copies). By special arrangement it has been translated and published abroad in many different languages. It is still going strong. In the foreword the author said "This primer is an attempt to develop in simple terms the principles governing concrete mixtures and to show how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete." 46 pages, 25 cents (cheaper in quantity)

For further information about ACI Membership and Publications (including pamphlets presenting Synopsis of recent ACI papers and reports) address:

AMERICAN CONCRETE INSTITUTE 742 New Center Building Detroit 2, Michigan

[To facilitate selective distribution, **separate prints** of this title (42-13) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than May 1, 1946.]

Title 42-13 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

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Vol. 17 No. 4 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **February 1946**

Maintenance of Heavy Concrete Structures
Minnesota Power & Light Company Practice*

By CLAY C. BOSWELL†
Member American Concrete Institute
and ALBERT C. GIESECKE‡

SYNOPSIS

The practice of The Minnesota Power & Light Company in repairing and restoring a concrete dam is described and illustrated and comparisons made with a much older structure, which has had no repair cost because construction methods were better.

The severity of the climate in the area of this company's operations produces a less frequent, but more severe type of frost damage, than that which occurs in a milder climate in the presence of a frequently reoccuring "cycle of freezing-thawing", such as is usually associated with concrete disintegration problems.

It is the deeper penetration of frost, into such concrete structures as are vulnerable to its damaging effects, which forms important reason for the practice developed by the Minnesota Power & Light Company, and which lends substantial meaning to the term "heavy concrete repair slab" as pictured in this article.

The practice of this company in dealing with problems of this type dates back to 1926 when the first major work of similar character was done. A number of comparable applications have since been made. Ages of two major applications are approaching 18 years, and results to date are most satisfactory.

In no sense is it the author's suggestion that the methods herein described be adopted in toto for other conditions; instead it is his conviction that conditions prevailing at each situation will require local decisions as to—

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1. Design of details of slab and reinforcing.
2. Design of concrete mix.
3. Spacing and detail of construction joints.

FOND DU LAC HYDRO ELECTRIC STATION DAM

If proof were wanted as to whether variations in technique used by the builder can make the difference between a *concrete dam of good quality*, as against one in which maintenance is a major item, it would be available by the simple process of inspecting the Thomson Dam and the Fond du Lac Dam, both on the St. Louis River, Minnesota, which, at this date, are 39 and 21 years old, respectively.

The excellent condition of the Thomson Dam was recognized in 1928 when it was described by H. C. Ash, in the A C I *Proceedings*, V. 25 of 1929.

Such lack of erudition as may have hindered the efforts of the builders of Thomson Dam, was in ample measure offset by diligent application of simple fundamentals. They used clean aggregate; mixed aggregate as densely as possible by combining pit gravel and lake gravel; they cleaned days-work planes, and, in the words of Mr. Ash "dry concrete requiring much tamping, and free water on the surface of placed concrete, were both discouraged with emphasis." The concrete, according to Mr. Ash, approximated proportions of 1:1.7:4.3.

The Fond du Lac concrete was crushed stone and pit sand, both of which were clean and sound. The mix was generally 1:3:6 and most concrete was placed by a system of chutes. Good practice was violated, principally in the wide variation and excessive use of mixing water, with consequent gross segregation which now is the obvious nucleus of most of the weakness requiring remedy within the decade ending approximately as the structure reaches the end of its first 25 years of service.

The 39-year-old Thomson Dam is in essentially a perfect state of preservation with no concrete maintanence expense "aft" or "forward." The 21-year-old Fond du Lac Dam, however, will, before it is 39 years old, have involved substantial maintenance costs in addition to those already incurred.

The Fond du Lac maintenance problem

Signs of impending concrete maintenance problems were seen as early as 1928, when the dam was but 4 years old, at which time disintegration was, in minor degree, noticeable in the taintor gate waterways. This was then, however, considered "*of no immediate concern.*" Greater concern was felt over the condition of two gate piers. Fig. 1, on close examination, will reveal that substantial evidence of disintegration was

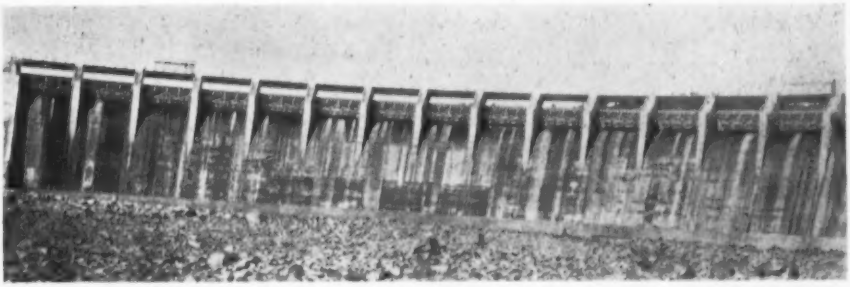

Fig. 1 (above)—Downstream face of Fond du Lac Dam showing taintor gates numbers 4 to 17 inclusive. September, 1928

Fig. 2 (right)—Downstream face of Fond du Lac Dam in area of gates 8, 9 and 10. June 1936

visible on the downstream face of the dam, even when viewed from several hundred feet away.

Condition in 1936

Fig. 2 shows the condition near taintor gates 8, 9 and 10 in June 1936. Also, in October, careful field inspection revealed taintor gate waterway spalling as shown on Fig. 3. The lesser depth of spalling in taintor gate 16, and at several adjacent ones, is accounted for, at least in part, by the less frequent discharge through these gates as compared with those near the middle of the dam.

At this point, records of original construction were reviewed and it was found that concrete specifications were complied with, except as to

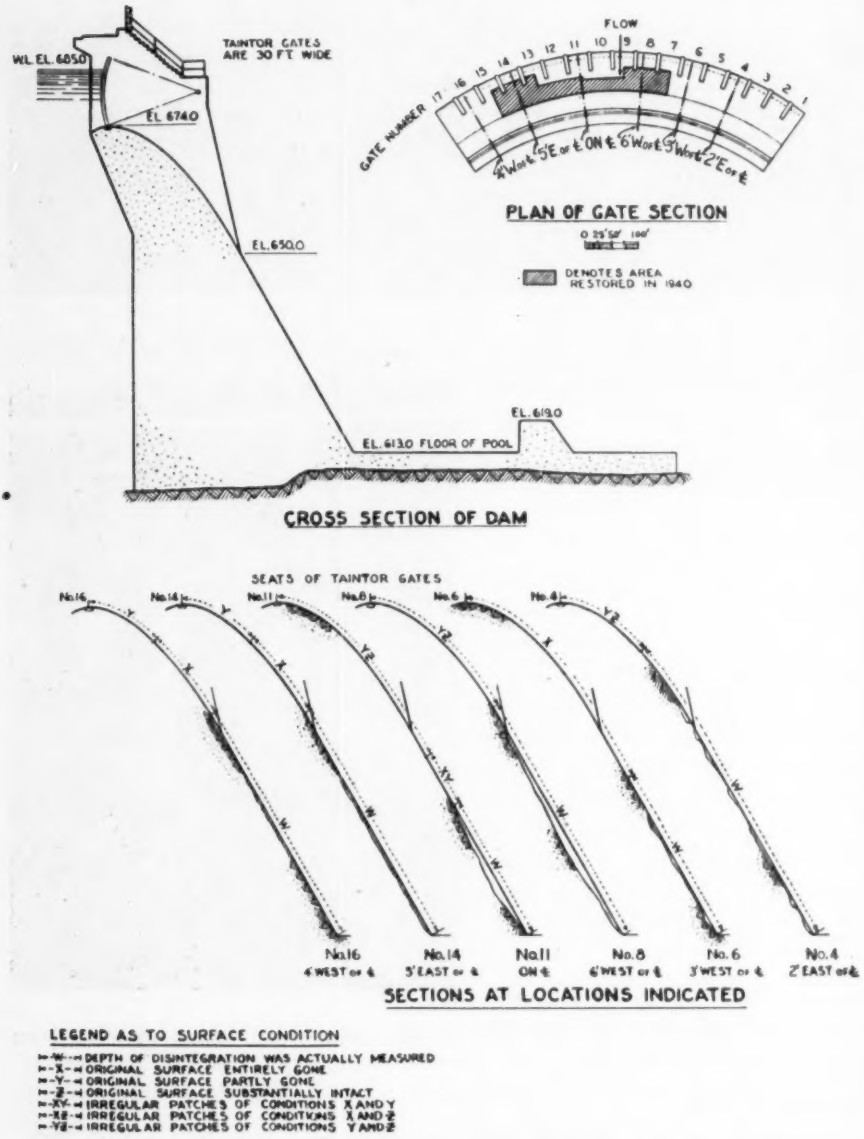
WL EL. 6050
TAINTOR GATES ARE 30 FT WIDE
EL. 6740
EL. 6500
EL. 6190
EL. 6130 FLOOR OF POOL
CROSS SECTION OF DAM
FLOW
GATE NUMBER
17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1
2'West of ℄
5'E of ℄
ON ℄
6'West of ℄
9'West of ℄
2'E of ℄
PLAN OF GATE SECTION
0 25' 50' 100'
DENOTES AREA RESTORED IN 1940
SEATS OF TAINTOR GATES
No.16
No.14
No.11
No.8
No.6
No.4
No.16 6'WEST of ℄
No.14 3'EAST of ℄
No.11 ON ℄
No.8 6'WEST of ℄
No.6 3'WEST of ℄
No.4 2'EAST of ℄
SECTIONS AT LOCATIONS INDICATED
LEGEND AS TO SURFACE CONDITION
W — DEPTH OF DISINTEGRATION WAS ACTUALLY MEASURED
X — ORIGINAL SURFACE ENTIRELY GONE
Y — ORIGINAL SURFACE PARTLY GONE
Z — ORIGINAL SURFACE SUBSTANTIALLY INTACT
XY — IRREGULAR PATCHES OF CONDITIONS X AND Y
XZ — IRREGULAR PATCHES OF CONDITIONS X AND Z
YZ — IRREGULAR PATCHES OF CONDITIONS Y AND Z

Fig. 3—Part record of inspection of downstream face of Fond du Lac Dam in October, 1936

Fig. 4—Downstream face of Fond du Lac Dam in area of first concrete restoration. October, 1940

lavish use of mixing water; compressive strength tests were made as the work progressed, and these should have been warning enough to call for corrective measures. The chuting system of placing concrete was the occasion for excessive and varying moisture contents, and this resulted partly in defective conditions throughout large areas, and elsewhere in lenses of poorer concrete (and even sheet laitance) interspersed inside masses of somewhat better quality.

Repair program

Factors outside the Fond du Lac Dam were largely responsible for adopting the plan to which the job readily lent itself, namely, dividing the repair job into 3 blocks, one each for 1940, 1945 and 1948. This program permitted the work to be kept within the capacity of the company's repair crew, staggered with the periods in which similar work elsewhere had to be done. Further it accommodated in best possible degree, the coordination necessary between work of this kind and the requirements of power production.

1940 repair work

The area restored in 1940 is indicated on Fig. 3, as well as in Fig. 4, which shows the forms still in place at taintor gates 14 and 15, and in the right foreground, the interspersion of weaker lenses in otherwise fairly sound concrete. In the distance, (as well as at the upper right corner), the generally poor areas forming parts of subsequent repair blocks is shown.

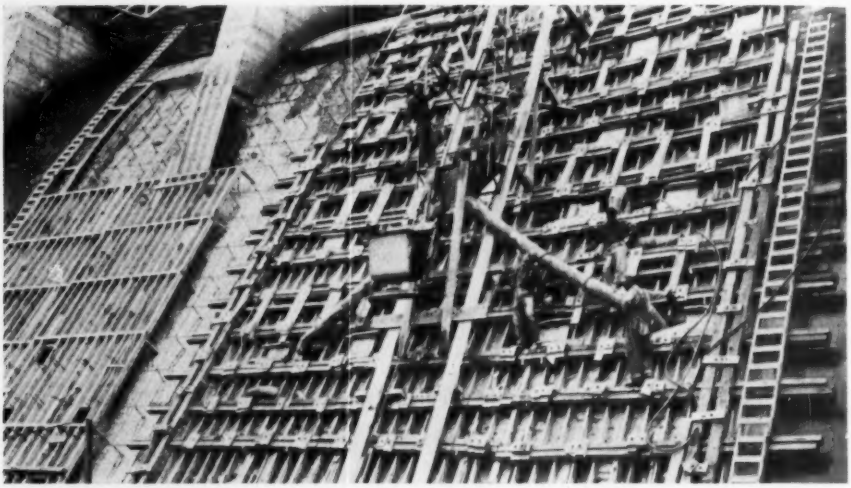

Fig. 5—Downstream face of Fond du Lac Dam showing details of construction. October, 1940

Methods of dealing with similar problems, developed on other structures as much as 14 years earlier, were applied at Fond du Lac in 1940, and at this date, no indication of failure has manifested itself. While necessarily the remedy involves a cover over large areas, the cover is in no sense a "veneer" because it is far too heavy to be so designated.

Detail of heavy concrete repair slab

The concept of a "heavy repair slab" is descriptive of the substantial thickness of reinforced concrete which we use to treat such a surface. The slab is usually specified to have at least 24 in. average thickness and well reinforced with reinforcing connected mechanically by hooks to the dowel rods, which, in turn, by wedge and, or grout, provide anchorage to the underlying concrete at depth beyond the reach of frost penetration.

At Fond du Lac the vertical construction joints in the repair slab were spaced on the center line of each taintor gate. This placed a repair slab joint at each of the original structure joints which occurred at center lines of alternate gates. The placing of concrete was carried from the bottom to the very top of each section as a continous operation.

The new concrete is deposited on surfaces generally scrubbed by a heavy spray of high pressure air and water; then, shortly before concrete is placed, a grout wash is applied. All concrete is of clean, sound aggregate, of standard portland cement (without admixture or air entraining agent) and water sufficient to produce the minimum slump consistent with placing the concrete without honey-combing. Our general practice is to use about 6 to 6½ bags of standard portland cement per

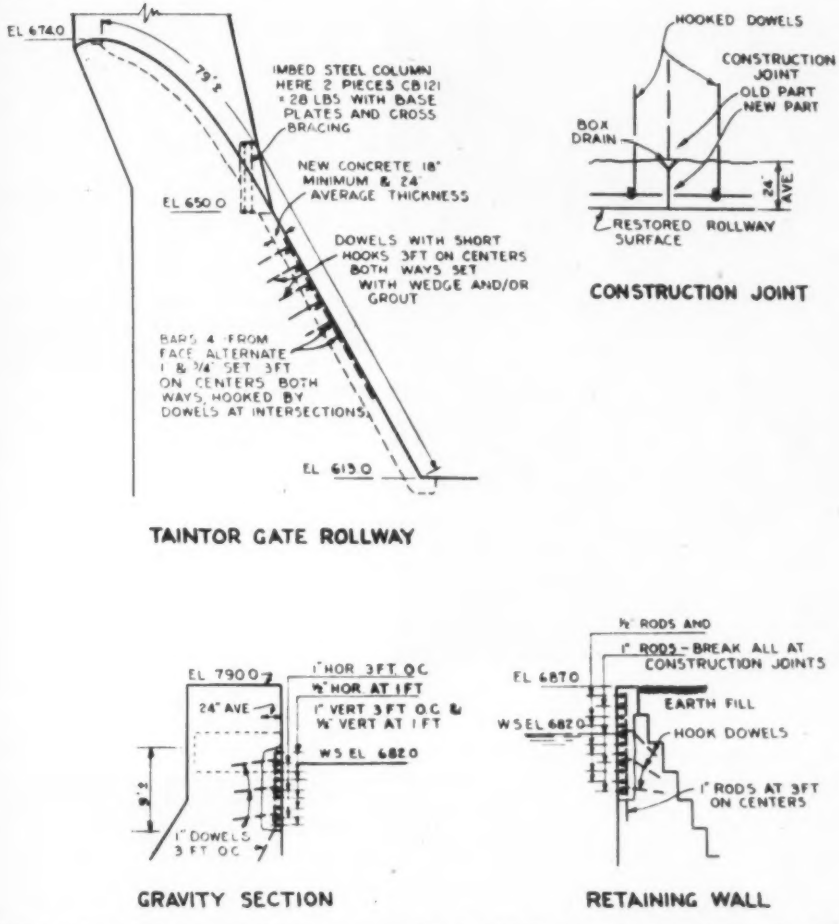
EL 6740
79'-5"
IMBED STEEL COLUMN HERE 2 PIECES CB121 = 28 LBS WITH BASE PLATES AND CROSS BRACING
NEW CONCRETE 18" MINIMUM & 24" AVERAGE THICKNESS
EL 6500
DOWELS WITH SHORT HOOKS 3FT ON CENTERS BOTH WAYS SET WITH WEDGE AND/OR GROUT
BARS 4" FROM FACE ALTERNATE 1 & 3/4" SET 3FT ON CENTERS BOTH WAYS, HOOKED BY DOWELS AT INTERSECTIONS
EL 6130
TAINTOR GATE ROLLWAY
HOOKED DOWELS
CONSTRUCTION JOINT
OLD PART
NEW PART
BOX DRAIN
24" AVE
RESTORED ROLLWAY SURFACE
CONSTRUCTION JOINT
EL 7900
1" HOR 3FT O.C
1/2" HOR AT 1FT
24" AVE
1" VERT 3FT O.C & 1/2" VERT AT 1FT
W.S EL 6820
9'-2"
1" DOWELS 3 FT O.C
GRAVITY SECTION
1/2" RODS AND
1" RODS - BREAK ALL AT CONSTRUCTION JOINTS
EL 6870
EARTH FILL
W.S EL 6820
HOOK DOWELS
1" RODS AT 3FT ON CENTERS
RETAINING WALL

Fig. 6—Drawing from specifications governing the 1940 concrete restoration at Fond du Lac Dam

Fig. 6a, upper left; 6b, upper right; 6c, lower right; 6d, lower left

cubic yard and internal vibration is resorted to wherever it can be applied. In curing green concrete, the usual precautions of applying ample moisture are observed for from six to eight days.

Fig. 5 shows many items of interest. At the extreme right edge, new concrete from which forms have already been removed is visible. Next is the large area in which concrete is being placed, while on the left is an area which is being readied for concrete, with panel forms covering a portion of the area, and leaving to view the arrangement of dowels and reinforcing bars, as well as giving indication of thickness of the concrete slab being provided. Fig. 6 shows the details in drawing form.

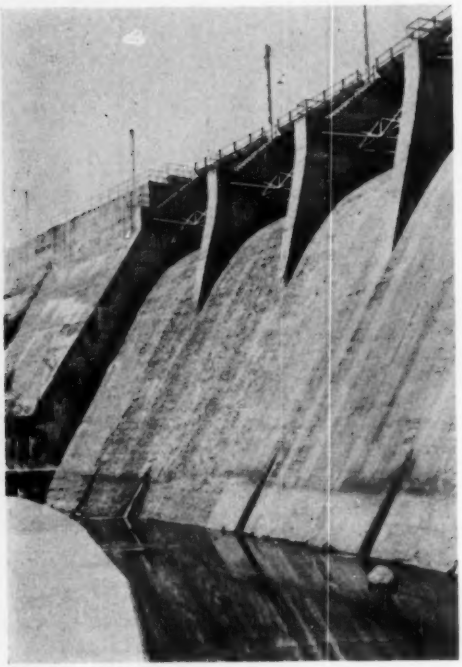

Fig. 7—View of 1940 concrete in the right foreground and of a disintegrated area beyond the center line of gate 15, which is scheduled for 1945 restoration. July, 1945

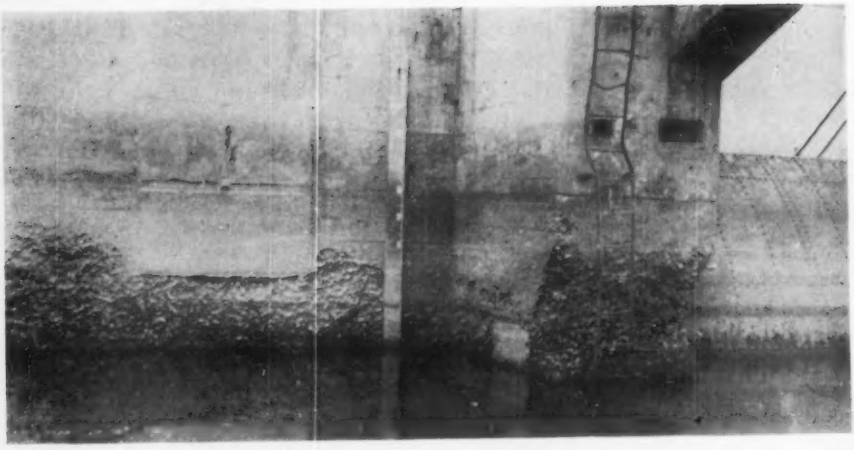

Fig. 8—Typical pond face disintegration on vertical walls at Fond du Lac dam; trash gate pier to right of staff gage. September, 1941

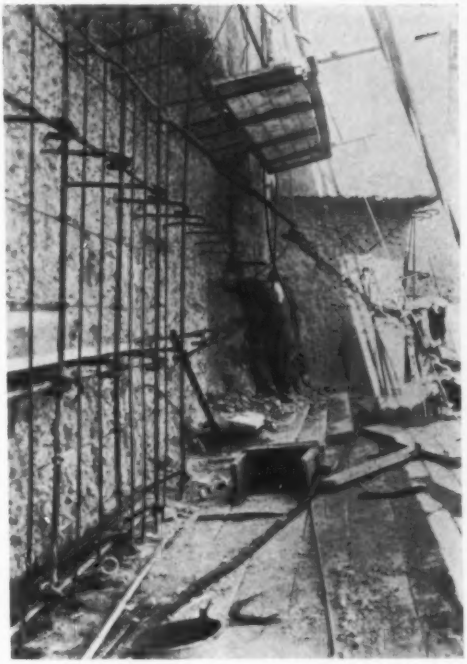

Fig. 9—Typical pond face restoration, showing in profile, the pier appearing on Fig. 8.

Fig. 6a shows the arrangement of dowels and reinforcing steel. It also indicates the position and manner of placing steel columns at the heel of the taintor gate pier. Such a steel column is partly visible behind the incompleted forms in Fig. 5.

Fig 6b shows detail at location of original construction joints.

1945 repair work:

Work scheduled for 1945 embraced downstream face areas, as well as pond face items. Fig 6c shows details of restoration of the forebay retaining wall, and Fig. 6d shows manner of dealing with water line disintegration.

Fig. 7, at the left, shows an area scheduled for treatment in 1945, and at the right, a large part of the 1940 work. The dividing line between these two areas is the center line of taintor gate 15.

Fig. 8 shows water line disintegration suffered at the pond face. This particular view shows the trash gate pier just to the right of the staff gage, and the manner of treatment given it is indicated in Fig. 9 which shows the same trash gate pier in profile.

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Two Special Methods of Restoring and Strengthening Masonry Structures*

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Members American Concrete Institute

SYNOPSIS

Structures and foundations damaged by weather, erosion, scour, or settlement have been restored and strengthened by ingenious methods involving the pumping of cement-base stabilizing material into small interstices and the filling of larger spaces by aggregate which is then embedded in the stabilizing material under pressure. Herein are described several applications of the methods to various structures including bridge piers and abutments, reservoirs, dams, and underwater construction.

INTRODUCTION

It has been well said that greater engineering skill is required to restore an old structure than to build a new one. Certainly restoration challenges the engineer and calls for a high degree of experience and ingenuity. However, it is the only solution to many problems where the use of the structure must not be interrupted and where the cost of removal and replacement would be prohibitive.

Many cut-stone structures on this continent are now almost a century old, and many concrete structures are about a half-century old. As a class these structures are durable and stable, but one need not go far to find many that are in need of treatment. Some are defective because of faulty design or construction resulting in porosity, honeycombing, laitance, or cracking. Others, even if well built, are in unsatisfactory condition because of severe weathering, leaching, cracking, or exposure to fumes or other corrosive agents. Foundations have been subject to scour and to uneven settlement. In many cases the live loads

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have increased far beyond the original design loads, and new uses of the structure have been developed.

Special attention to these conditions has long been given by the construction organizations whose work is described herein, and hundreds of structures have been reconditioned through the use of two special techniques developed not only by field experience but also by extensive laboratory research. While these are proprietary processes, their application has been so widespread and so consistently successful that a factual account of their general nature and typical accomplishments will be of interest to all who have the responsibility for the maintenance and repair of masonry structures.

In order to avoid repetition in the descriptions of the work on individual structures, it is desirable first to explain the two general processes, intrusion and prepacking.

INTRUSION

Briefly, the intrusion process of stabilizing a structure or foundation consists in drilling holes at intervals and injecting an intrusion mixture which consists of portland cement, a powdered mineral filler, an intrusion aid, water, and in some cases a fine sand. The materials and proportions are necessarily varied to meet the particular conditions, within limits which are established by laboratory test and field experience. Features of the process are the characteristics of the intrusion mixture, special fittings and connections for intrusion, the sequence of solidifying various portions of the mass, and the determination of adequate penetration.

The mineral filler used in the intrusion mixture is a finely divided siliceous material called Alfesil. In the fresh mixture it tends to prevent agglomeration of the grains of portland cement and thus increases their effectiveness. In the hardened mixture, it combines with lime liberated by the hydrating portland cement to form calcium silicates which are relatively insoluble and which contribute to strength. The effect of this puzzolanic material becomes evident in the considerable development of strength at later ages, after a slow start.

The intrusion aid, or agent as it is commonly called, is used primarily for the purpose of increasing the pumpability or flowability of the mixture through the extremely small crevices and voids which must be traversed to secure thorough penetration of the structure by the solidifying material. The agent also assists in dispersal of the cement grains, and together with the filler it reduces settlement and eliminates early shrinkage of the mixture after it reaches its final location in the void spaces. Numerous test specimens cut from structures which have been

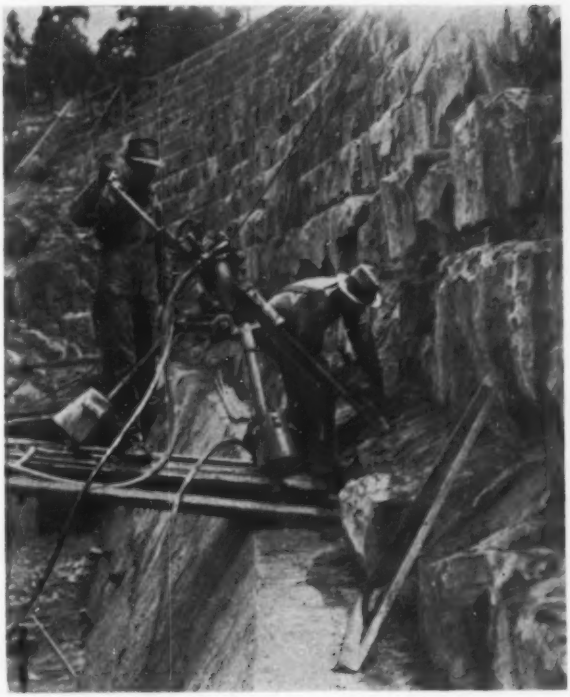

Fig. 1.—To prepare for intrusion, holes are drilled throughout structure and foundation.

intruded show that the spaces remain completely filled, without the partial separation from the walls and top of the cavity that is often observed in the case of portland-cement grouts.

Each structure presents distinctive problems, but in general the process of intrusion involves the following steps. Holes are drilled at various intervals and to various depths as required by the condition of the structure (Fig. 1). Care is of course taken not to damage the structure. Before a given section is treated it is tested under water pressure in order to determine whether the drilling is adequate and to select the proper consistency of the intrusion mixture.

The intrusion mixture is thoroughly mixed by mechanical stirring and agitation until a smooth slurry is obtained (Fig. 2). Where a high degree of penetration into small voids and passages is required, the mixing time may be as much as 20 minutes. The mixture is agitated continually in order to keep it uniform.

The intrusion mixture is injected by pumping under pressure sufficient to secure thorough penetration of the structure without displacement of structural parts (Fig. 2). The sequence of filling the various holes and, if necessary, the various depths in a given hole, is such as to expel the water and air from the cavities ahead of the intrusion mixture. When no

Fig. 2. — Materials from platform (center) are measured into mixing tanks. Piston pumps at left force intrusion mixture into structure.

Fig. 3.—A special expanding connection called an intrusion insert is used to connect pressure hose to hole.

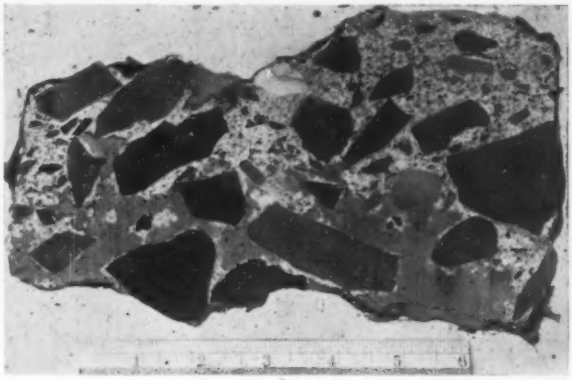

Fig. 4.—Piece of concrete cut from disintegrated tunnel lining 70 ft. from nearest point of intrusion. Note penetration of intrusion material (solid gray) into old structure.

additional material can be intruded immediately, pressure is maintained long enough to insure any additional penetration that may be made possible by slow flow of the mixture into cavities. The thoroughness of penetration at any given hole is judged by the inflow in relation to conditions observed during the drilling, by the pressure at which intrusion stops, and by the showing of intrusion mixture at distant cracks or other openings. When necessary, adjacent openings are calked to prevent loss of intrusion material. A special expanding connection has been developed to connect the pressure hose quickly and tightly to the drilled holes (Fig. 3).

The results obtained by intrusion become evident in the performance of the structure. If the structure is one which is exposed to water under pressure, the results can be observed visually at once. The amount of material intruded is of course one measure of solidification. Surprisingly large amounts can often be injected into structures which on the surface appear to be fairly sound (Fig. 4). Test cores may be taken to prove the completeness of penetration and to determine the strength.

Tests for strength and other physical properties of the intrusion mixture itself have been made by a number of laboratories. It must be emphasized that the mixtures are not standardized but are proportioned to meet the requirements of the particular job, and that any strength equal to that of comparable concretes can be produced. In one series of tests, 2-by 4-in. cylinders and standard tensile briquets were molded from a mortar which contained equal parts by weight of intrusion paste and sand, with a ratio of water to cement plus Alfesil of 0.47 by weight. The specimens were standard-cured. The following strengths were obtained.

	7 day	*28 day*	*3 mo.*	*1 yr.*
Compressive strength, psi	2510	4200	6860	9090
Tensile strength, psi	330	465	505	590

Comparative tests of intrusion pastes with portland-cement grout of equal consistency disclose (1) that the bleeding of the intrusion paste is negligible whereas under the same conditions the portland-cement grout bleeds as much as 4 per cent of water by volume; (2) the intrusion paste quickly passes through glass tubes filled with coarse sand, which cannot be penetrated with portland-cement grouts of corresponding consistency; and (3) the volumetric shrinkage during the first 24 hours is far less for the intrusion paste than for the portland-cement grout.

To summarize, the intrusion mixture maintains the solids in suspension, flows easily and without plugging, is low in setting shrinkage, and develops in the cavities a dense strong paste tightly bonded to the walls.

PREPACKING

Prepakt is concrete made by packing the forms with coarse aggregate and then pumping in a cement-base intrusion mixture to fill the voids under pressure. It is used both for restoration and, in special cases, for new construction.

One principal purpose of placing the aggregate in advance is to bring the pieces of aggregate into contact or near-contact, without the clearance which is required for plasticity in ordinary concrete. By this arrangement, even if the intervening mortar were normal in drying-shrinkage characteristics, overall shrinkage of the concrete would be relatively low because the pieces of coarse aggregate are in contact with one another. When the coarse aggregate is suitably graded the void spaces are relatively low, with correspondingly low requirement of mortar and therefore cement. The low cement content, together with the slow-hardening characteristics of the intrusion mixture, make for low and slow heat generation, which is of importance in mass concrete.

Packing the aggregate in advance also facilitates placing in difficult locations, and permits inspection before concreting in order to assure complete filling of the forms.

As in the case of intrusion mixtures, the strength of Prepakt concrete is made to meet the job requirements. The average compressive strength of 32 Prepakt concretes for which records are conveniently available, tested in the form of 6- by 12-in. cylinders, was 2,200 psi at 7 days, 3,540 psi at 28 days, and 4,330 psi at 3 months. Some of these concretes attained a compressive strength of 3,200 psi at 7 days and 6,700 psi at 3 months. The drying shrinkage of Prepakt concrete is less than that of ordinary concrete of the same cement content, and the resistance to cracking is correspondingly high. Resistance to weathering has been established as satisfactory not only by accelerated laboratory tests but

also by field experience under several years of exposure. As in regular concrete, resistance to freezing and thawing may be markedly improved through the use of air-entraining agents.

The bond strength of Prepakt concrete to regular concrete, tested in the form of beams jointed at midspan, was found to be about 65 per cent greater than that for regular concrete cast against regular concrete; in fact, it was about 70 per cent of the strength of corresponding unjointed beams of regular concrete. In freezing-and-thawing tests of Prepakt concrete cast against air-entrained regular concrete, the joint between the two portions remained intact even after severe disintegration of the concretes themselves had taken place.

The general process of restoration with Prepakt is illustrated in Fig. 5a to 5c. Defective concrete of the structure is chipped out, a form is applied, and the space behind the form is packed tightly with coarse aggregate. Intrusion mixture is pumped in and, under pressure, not only fills the spaces within the prepacked aggregate but also penetrates into the pores of the underlying surface and into connecting cracks and void spaces. The cementing to and stabilizing of the underlying mass is a distinctive feature of this method of repair.

New construction with Prepakt is performed in a similar manner, with modifications as necessary. For example, in a larger mass the intrusion inlets which are placed at intervals horizontally and vertically may be perforated pipes embedded within the aggregate; and the intrusion mixture may be injected successively at various elevations. For underwater work it is usually not necessary to unwater the structure. Tests in field and laboratory have shown repeatedly that the rising mass of intrusion mixture displaces the water without mixing with it, and that satisfactory strengths and densities are uniformly obtained. Fig. 6 shows one laboratory test in progress; intrusion mixture is being forced in at the lower left edge of the form and is rising through ½ to 1½-in. aggregate previously submerged in water. In comparable tests with portland-cement grout, the zone of demarkation between grout and water is cloudy, and segregation and dilution of the grout occur.

APPLICATIONS

On many piers of the older railway bridges, surface disintegration has occurred due to weathering particularly at the water (and ice) lines. Fig. 7a and 7b show restoration in a typical case. First the foundation rock and the masonry base of the pier were intruded. Then the interior, which was filled with loose rock, was solidified as well as the mortar joints. Defective portions of the concrete pedestal cap and the facing stones were removed and replaced with Prepakt concrete; it was

necessary to cast a Prepakt blanket completely around the lower portion. In another structure the pier footings were extended downward and outward with Prepakt concrete, providing not only a greater bearing area but also a bearing on firmer ground at the lower elevation.

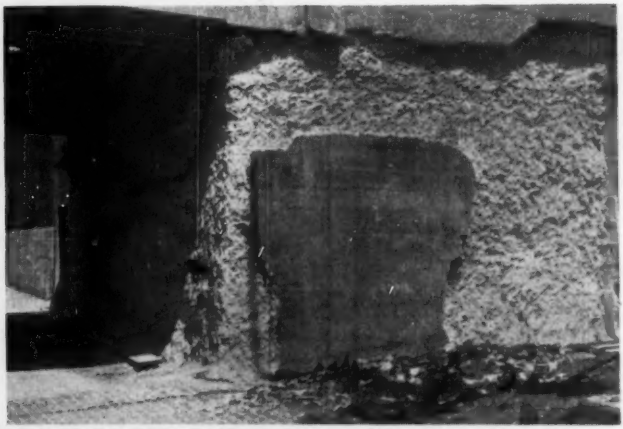

Fig. 5a.—To prepare for restoration with Prepakt, old concrete is trimmed away to sound material.

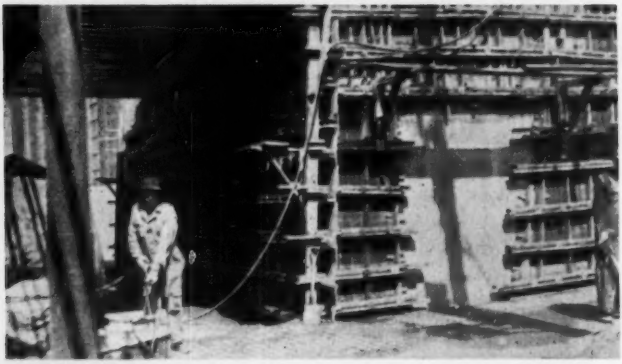

Fig. 5b.—Forms are packed with coarse aggregate, and intrusion mixture is pumped in.

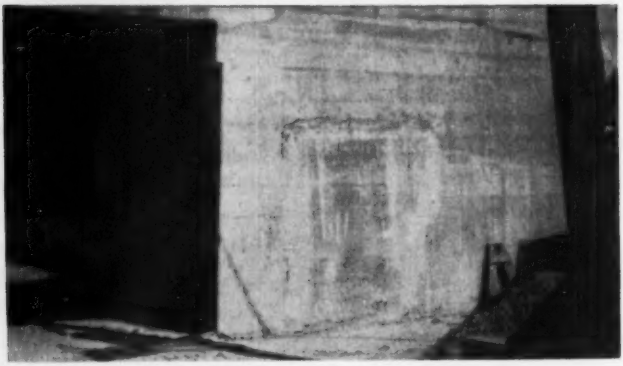

Fig. 5c.—Finished surface, before cleaning.

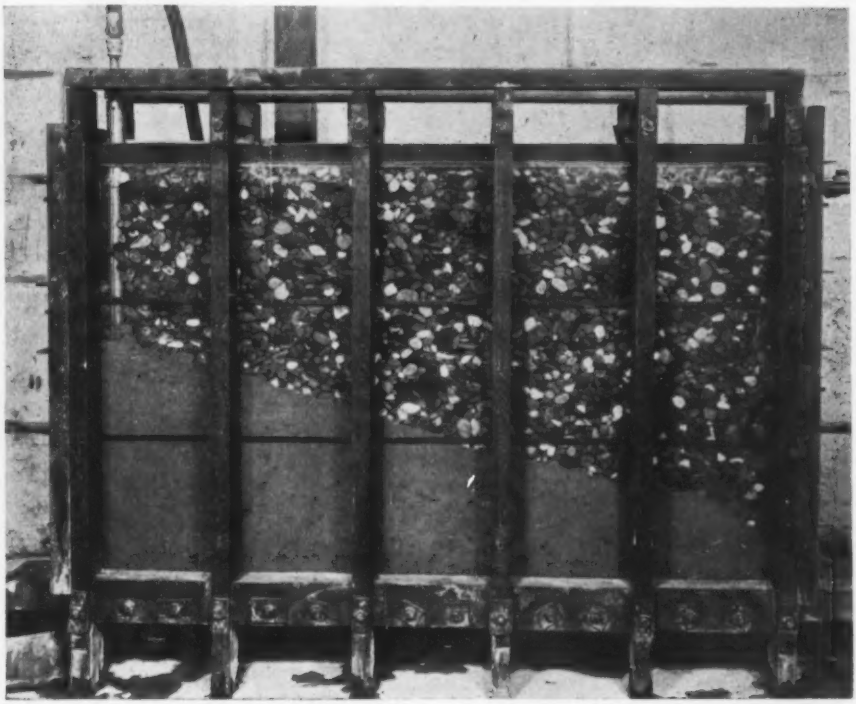

Fig. 6.—Laboratory test of placing Prepakt concrete under water. Note sharp line of demarkation between rising intrusion mixture and water being displaced.

During floods it is not unusual for mud or gravel to be washed from under piers which rest on piles. If the amount of material removed is considerable, the lateral support to the piles is lessened to such an extent that the structure weaves noticeably under load. One instance in which this condition was corrected by intrusion is illustrated in Fig. 8. Holes were drilled down through the footings, mud and sand were flushed out, and the space was filled with intrusion mixture, which also spread around the pier and stabilized an area greater than the original. The holes were extended further downward and were intruded to form a solid wall to prevent future scour. Still further down, the existing layer of coarse sand and gravel was flushed out and solidified by intrusion. The approximate final limits of intrusion, as determined by drilling, are indicated on the drawing. In stabilizing the piers of other bridges, in some cases the opening beneath the pier has been so large that it was filled with riprap or with prepacked aggregate in order to reduce the amount of intrusion mixture required.

Abutments and wing walls are restored in a manner similar to that previously described for piers. A typical case is illustrated in Fig. 9a

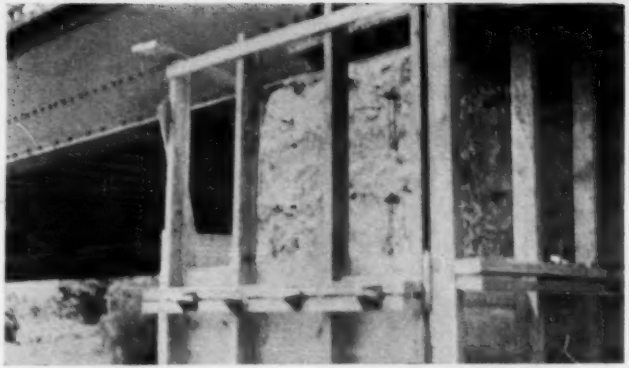

Fig. 7a—(top left) Disintegrated concrete of pedestal block has been chipped away, and forms for Prepakt concrete are being erected

Fig. 7b—Disintegration of cut-stone facing was so severe at water line that Prepakt concrete blanket was built entirely around pier.

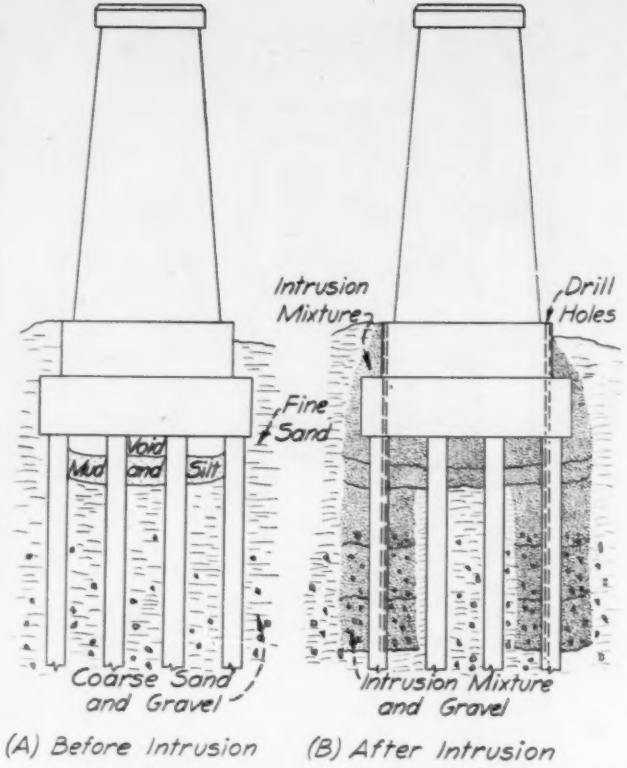

Fig. 8—Stabilization of railway bridge pier on piles.

Fig. 9a (above)—Several courses of stone were removed from this old abutment, and a new bridge seat of Prepakt concrete was cast.

Fig. 9b (top right)—Prepakt concrete has replaced the old bridge seat and disintegrated facing stones of the old wing wall. During construction, bridge girders were supported on bent shown at left.

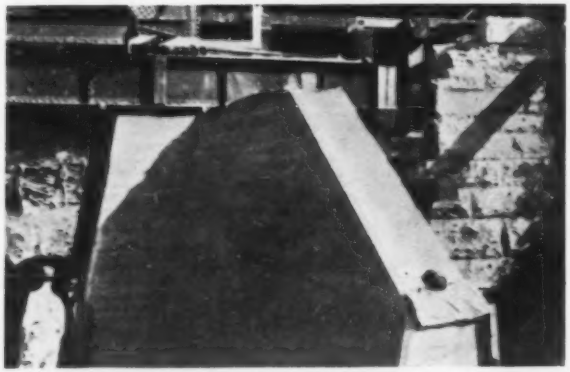

Fig. 10 (bottom right)—New construction with Prepakt. Forms for entire abutment and wing wall packed with local aggregate and intruded.

and 9b. In addition to the work shown, the entire interior of the masonry and joints was solidified by intrusion. In another case an entire abutment was built with Prepakt concrete which could be cast more economically with plant already at the site than by bringing in a conventional plant and casting ordinary concrete (Fig. 10).

A common type of early construction of small single-arch bridges consisted of cut-stone masonry to the spring line and brick masonry in the arch. Overcoating of the disintegrated masonry would lessen the opening of the waterway. Several bridges of this type have been restored by removing the stone in alternate lengths and casting Prepakt concrete to the original line of the structure (Fig. 11a and 11b). Solidification of the Prepakt under pressure assured tight contact and prevented settlement of the arch. In one case, the arches of a stone bridge were deepened over their entire area with a Prepakt-concrete arch which because of the pressure filling was able to share the load without settlement of the arch.

Restoration of the piers of a long railway bridge under heavy traffic and severe underwater conditions was accomplished by the intrusion and Prepakt methods in 1941-1943.* The piers, 70 years old, had been built by sinking watertight timber caissons in place and then building a concrete base and cut-stone pier on each base. Around some of the piers had been placed open-bottom caissons which were filled with riprap. Deterioration of the outer caissons eventually allowed much of the riprap to escape; in some cases damage to the timber of the central caisson had permitted erosion and disintegration of the concrete foundation; and the cut-stone portion of the pier was in bad condition particularly at the joints. At some of the piers, the water was as deep as 50 feet, and the current was as swift as 12 miles per hour. In spite of these adverse conditions, the entire restoration was accomplished at a cost estimated to be less than that for the construction of one new pier. In the upper portion of the piers, which were originally faced with ashlar masonry and filled with rubble masonry set in lime mortar, the volume of intrusion material pumped in was up to 12 per cent of the overall volume.

Another outstanding renewal, that of the 24 piers of a railway bridge 85 years old, was accomplished at a cost estimated to be one-tenth of the cost of replacement.† In no case was it necessary to unwater the piers; only a row of sheet piling was driven across the nose of the pier in order to slow the current and permit proper inspection and work by divers. Figure 12 is a winter view under this bridge.

*"Substructure Repairs Under Difficult Underwater Conditions," *Roads and Bridges* (Toronto), Nov. 1943, pp. 36-40, 65-69.

†"Rejuvenates 85-Year-Old Bridge Piers," C. P. Disney, *Railway Age*, June 16, 1945, pp. 1049-1051.

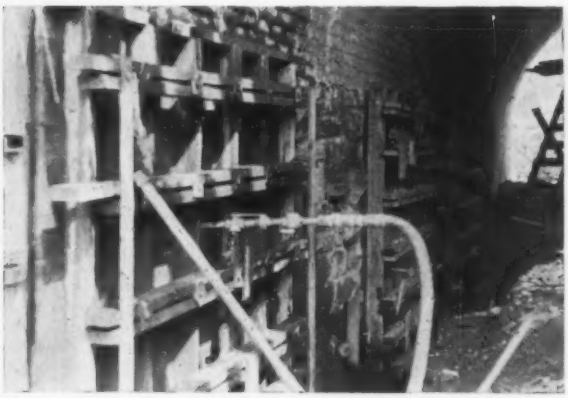

Fig. 11a (left)—Disintegrated stone masonry in bench walls of this single-arch railway bridge was removed in alternate 5-foot sections and replaced with Prepakt concrete.

Fig. 11b (right)—Second stage of replacement—Prepakt concrete sections permit full flow section and provide smoother surface than original construction.

The lining of several tunnels has been solidified and stabilized by the intrusion process, with Prepakt concrete used whenever required to replace displaced or damaged sections. One of the earlier jobs is described in *Western Construction News*, Feb. 1939, pp. 35-38 and in *Railway Age*, March 4, 1939, pp. 373-376.

Fig. 12.—One of two long railway bridges for which the piers were restored by intrusion and prepacking. Severity of ice conditions causing disintegration is apparent. Scaffolding for work interrupted by winter conditions is still on nearest piers.

A reservoir, shown in Fig. 13a and 13b, had been built in sandstone excavated to a depth of 14 feet and on the fill formed by the excavated sandstone to an additional depth of 14 feet. The sides were on a slope of 1 to 1. The entire area of the cut and fill had been surfaced with a blanket of clay, on which had been cast a concrete slab 4 to 6 in. thick. Settlement of the fill was so serious that it was impossible to fill the reservoir more than half full, and much of the clay blanket was eventually washed away leaving large voids and cavities. This condition was corrected by intrusion through and under the blanket, and the reservoir was rendered stable and watertight.

A large concrete dam built in the days of long gravity chutes, wet mixes, and consequent segregation and laitance formation, is shown in Fig. 14. Vertical construction joints were spaced 25 ft. apart, and lifts were about 5 feet deep. Leakage of water through both types of joint was extensive. In order to stop the leakage through the vertical joints, holes were drilled at intervals along each joint, extending into the rock foundation. Intrusion was completed in the early spring, when the width of the joints was a maximum due to seasonal contraction of the concrete blocks. The horizontal construction joints were also sealed

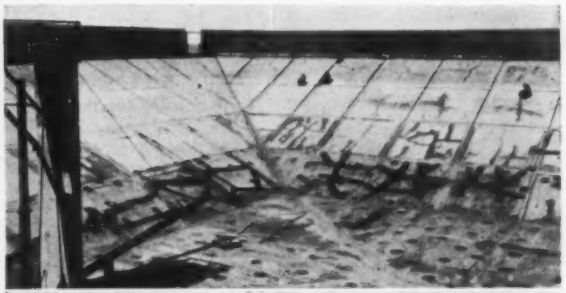

Fig. 13a.—Reservoir built half in excavation and half in embankment showed extensive leakage in spite of clay blanket.

Fig. 13b.—After intrusion to seal foundation and replace eroded portions of blanket, joints were sealed and entire surface treated with waterproofing compound.

Fig. 14.—Vertical and horizontal construction joints have since been sealed by intrusion. Downstream face will be restored later.

by drilling and intruding from the top, each laitance seam being thoroughly flushed out before intrusion.

One interesting development has been the solidification of rock-fill breakwaters and sea walls to prevent scour and break-up in heavy storms. Figure 15 shows a troublesome section of rock apron which has been stabilized by filling the spaces between the large stones with crushed-rock aggregate and then intruding the mass to form a five-foot apron and toe wall extending well below low tide. In a subsequent storm, much of the untreated rock section shown at the left of the photograph was washed out, whereas the treated section is still intact.

Inspection of a large concrete-lined spillway tunnel disclosed several eroded cavities in the invert. The largest of these breaks was about

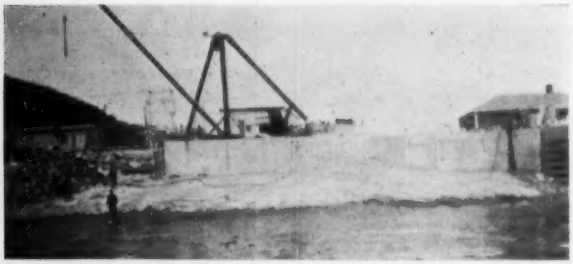

Fig. 15.—Section of sea wall subject to heavy tide and wave action has been stabilized by intruding to form a solid blanket 5 feet thick. Rock fill at left has since been washed out by storm, leaving the intruded apron intact.

112 ft. long and 33 ft. wide, with a maximum depth of scour of 36 ft. A description of the repair work by the Prepakt method is given in "Erosion Causes Invert Break in Boulder Dam Spillway Tunnel," by Kenneth B. Keener, *Engrg. News-Record*, Nov. 18, 1943, pp. 762-766. Quoting from the description, " . . . investigations indicated that such a method had definite advantages in that the high bonding strength of prepacked concrete, its low cement content and low temperature rise, small shrinkage and therefore less inclination to cracking, would more than compensate for its slowness in attaining strength. However, the primary reason for adoption of that process was the conclusion that the repairs could be made more economically than by regular concrete placing methods."

A unique design for bridge piers in deep water has been devised by C. P. Disney.* In this design, steel H-piles set in the bed rock are encased in a Prepakt-concrete pier. Sheet piling is driven to the outline of the finished pier, the enclosure is washed clear of all material except that which is suitable for inclusion in the structure, the remaining space is packed with aggregate, and the pier is intruded to form a monolithic whole. While the operation is not that of reconditioning as are those previously described, it illustrates the possibilities for alterations of, and additions to, existing structures.

The work of restoration by either method discussed herein is conducted by Intrusion-Prepakt, and new construction using the Prepakt method by the Prepakt Concrete Co., both with offices in Chicago, Cleveland, and Toronto.

*"Redesign of the Quebec Bridge," by C. P. Disney, *Roads and Bridges* (Toronto), Feb. 1943, pp. 17-22, 48.

[To facilitate selective distribution, **separate prints** of this title (42-15) are currently available from ACI at 50 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than May 1, 1946.]

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Laboratory Studies of Concrete Containing Air-Entraining Admixtures*

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Member American Concrete Institute

SYNOPSIS

The effects of the incorporation of each of nine different air-entraining admixtures in concrete were investigated by the making of a large number of batches of concrete under carefully controlled laboratory conditions. The results of tests on the plastic and hardened concrete specimens from batches made in parallel with and without each admixture are presented and discussed. An interpretation of the significance of the data and their application to the successful use of air entrainment in concrete is given.

INTRODUCTION

The practical advantages to be derived from the entrainment of well distributed minute spheroids of air in concrete mixtures were brought to the attention of the writer in 1939 by work performed by the Portland Cement Association on experimental pavements. Several field and laboratory investigations had been instituted by others during the previous decade with some vague idea of the benefit to the hardened concrete inherent in air entrainment. Search of the literature reveals a number of references to the use of fats, oils, and greases in cement earlier in this century as is indicated in the bibliography appended to this paper. Reference was found (2)‡ to the following specification for stucco as written by Marcus Vitruvius Pollio, the famous Roman architect, in the first century A.D.; "A mixture of well-hydrated lime, marble dust and white sand mixed with water, *to which mixture is added either hogs' lard, curdled milk or blood.*" The reference to the "blood" is sinister,

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(‡) The numbers in parenthese refer to the bibliography appended hereto.

but the reference to "hogs' lard" is illuminating in that it indicates that the practical use of the benefits of entrained air is not a twentieth-century discovery. This earlier work notwithstanding, the peculiar merit of purposeful entrainment of air in concrete has become generally known and made use of only in the past five years.

The data compiled by this laboratory from tests on field specimens and observations made in connection with the construction of numerous projects, using plain and air-entraining concrete, by the Corps of Engineers since April 1941 have been published (79). The results of studies by this laboratory of various forms of Vinsol resin interground with cement have also been published (30, 31, 45, 65, 71, 77, 81 and 82).

It is the purpose of this paper to present the results of the most recent laboratory studies of air-entraining admixtures which were authorized:

(a) To determine if the benefits were solely attributable to the agent used or to the air entrained or to a combination of both;

(b) To determine the most effective use of air entrainment and the optimum limits of such use, and

(c) To develop a specification for air-entraining admixtures and related methods of test.

The work was divided into an initial and secondary phase, the results of which are hereinafter reported in detail, and a tertiary phase, involving concrete containing large (6-in.) aggregate, which has not progressed sufficiently to warrant more than brief mention at the close of this paper.

INITIAL PHASE

Admixtures

It was decided that the investigation should not be limited to admixtures which had been interground with the cement, but should include those which are added at the mixer. For this purpose, the materials listed in Table 1 were included in the work.

TABLE 1—NATURE OF ADMIXTURES

Q	A combination of spent transformer oil and triethanolamine.
R	A neutralized form of gasoline insoluble resin of the pine tree.
S	R plus calcium chloride (non-proprietary).
T	Saponified beef tallow (non proprietary).
U	A combination of an alkali salt of a fatty alcohol sulfate and calcium lignin sulfonate.
V	A combination of R, calcium chloride and aluminum powder.
X	Paraffin-oil (non proprietary).
Y	A solution of calcium chloride in water containing a small amount of an unidentified organic material—not an air-entraining agent.
Z	A combination of fly ash, calcium chloride and calcium lignosulfonate.

NOTE: *In all cases the symbol P represents concrete without admixture.*

Preparation

The admixtures were prepared and used in the manner and in the amounts described below:

Admixture Q was used in the amount of 20 ml. per 100 lb. of cement or *0.015 per cent of the admixture solids by weight of cement.* The amount to be used, based on the weight of cement, was measured out by volume and then diluted with water to make 100 ml. of liquid to be added at the mixer. The amount of mixing water was decreased by 100 ml. when Admixture Q was used.

Admixture R. The proportions of the R solution were: resin powder: 249 g.; sodium hydroxide; 37.4 g.; water: sufficient to make two liters of the solution .

The *NaOH* was dissolved in a small amount of water and then added slowly to a suspension of the resin powder in water. Stirring was continued until neutralization of the resin was complete. The solution contained 0.1245 g. of resin per ml. of solution. It was desired to use *0.01 per cent neutralized resin by weight of the cement,* so 34.3 ml. of solution was required per bag of cement. From the weight of cement to be used in each batch of concrete, the volume of the solution required was computed, measured, and then diluted with water to a total volume of 100 ml. for addition to the batch. The amount of mixing water was reduced by an equivalent volume.

Admixture S. The R admixture was prepared in solution and measured out exactly as described above except that it was not diluted. Commercial calcium chloride (77 - 80 per cent $CaCl_2$) was weighed out in the amount of 1 per cent by weight of the cement. A volume of cold water was measured out, as follows:

$$\text{Vol. of water, ml.} = \frac{W}{94} \times (473.17 - R)$$

where : W = weight in pounds of cement used,

R = volume of R solution to be used, in ml. and

473.17 = number of ml. in 1 pint.

Hence the volumes were such that the volume of the R solution + volume of water would be 1 pint per bag of cement.

The cold water was added to the $CaCl_2$ and it was stirred and heated until solution was complete. The $CaCl_2$ solution was then allowed to cool. Immediately prior to adding to the mixer the $CaCl_2$ solution was added to the R solution and the mixture stirred. The mixture was "stringy" when added to the mixer. When this admixture was used the amount of mixing water used was decreased by an amount equal to the combined volumes of the R solution and the water added to the $CaCl_2$

(i.e. by 1 pint per bag of cement). The admixture S was used in the amount of 1.06 per cent by weight of the cement.

Admixture T. The semi-liquid beef tallow was weighed out in four 120-g. batches. To each batch was added a solution containing 50 g. of $NaOH$, 150 ml. of H_2O, and 100 ml. of alcohol. The mixture was then boiled slowly (refluxed) until saponification was complete. The saponified mixture was then added to a cold saturated solution of $NaCl$ in water to "salt-out" the soap. The soap was then separated from the impurities and the salt solution by vacuum filtration. The soap was dried at 95 C. to constant weight and pulverized by grinding in a mortar. The powdered soap was weighed out in the amount of *0.01 per cent by weight of the cement* and then approximately 1000 ml. of water was added to the soap. This mixture was heated and stirred until the soap was completely dissolved. The soap solution was then diluted with warm water to a volume of 3000 ml. When this admixture was used, the amount of mixing water was decreased by 3000 ml.

Admixture U was obtained as a dry powder and the amount to be used was measured by weight so that *1.06 per cent was added by weight of the cement* used (equal to 1 lb. per bag of cement, as required by the manufacturer). The admixture was added to the batch in the powder form following the addition of first 2/3 of the water and before the cement.

Admixture V (*not used in initial phase of work*). This admixture was developed by the writer during the progress of the work*. It was used in lieu of admixture S in all except the initial phase of the work and was added to the batch in the powder form in the amount of *1.06 per cent by weight of the cement.*

Admixture X. The paraffin oil (see list of properties below) was received as an oil in which precipitated lumps of paraffin were suspended. The mixture was heated on a water bath until the lumps were completely melted. An amount of the oil equal to *0.5 per cent by weight of the cement* was taken up in a graduated pipette and expelled in a thin stream into the weighed amount of cement. The amount of cement treated varied from 60 to 100 lb. per batch. The cement was then mixed in a modified Los Angeles Abrasion Machine (from which the shelf had been removed) for 500 revolutions with a weight of ¾-in. diameter steel balls equal to 4 times the weight of the cement. The cement was then removed from the drum, separated from the balls, and placed in 5-gal. cans which were sealed except for pressure-relief valves. The cans were placed in an oven, maintained at a temperature of 200 F. for two days, and allowed to cool with the oven for two days. They were

*Patent applied for.

removed from the oven and the cement weighed out for use in concrete on the day the concrete was to be made. This aging was stated to be necessary by the agency recommending the material.

PROPERTIES OF PARRAFIN OIL

Wax Content	20 to 25 per cent
Specific Gravity	0.850 to 0.857
Flash Point	230 F. to 260 F. (Cleveland Open Cup)
Cold Test	65 F. to 75 F.
Viscosity at 100 F.	43 to 48 (Saybolt Universal)

Admixture Y was purchased as a liquid from a local distributor and was added to the mixer in the amount of 1.5 gal. per cu. yd. (abs. vol.) of concrete as recommended by manufacturer for "mass" concrete. The amount to be added to a given batch was determined by weight so that for the following cement factors the per cent by weight of cement was, as follows:

C. F. (b. per cu. yd.)	Admixture, per cent
4.50	2.96
5.25	2.53
6.00	2.22

When admixture Y was used the amount of mixing water was decreased by an amount equal to the weight of solution added.

In view of the claim by the manufacturer that this material afforded integral curing, the concrete made with this material received no fog or water curing after molding.

Admixture Z was purchased as a dry powder from the manufacturer's commercial stock in August 1944 and the amount to be used was measured by weight so that *1.06 per cent was added by weight of the cement* used (equal to 1 lb. per bag of cement, as required by manufacturer). The admixture was added to the batch in the powder form following the addition of the first 2/3 of the water and before the cement.

Cements

It was found to be impractical to obtain several brands of commercially produced portland cement containing the desired amounts (based on air entrainment) of the various intergrindable admixtures (Q, R, T, and X) and, as certain admixtures of the non-interground variety were to be used, it was decided to add all of the admixtures at the time of mixing the concrete (except X, see description of use above). The cement used was a blend of equal parts of four brands of cement manufactured commercially to meet Federal Specifications SS-C-206a (A.S.T.M. C 150 Type II). Proper quantities of equal parts of each of the cements were added to each batch of concrete at the time of mixing.

Aggregates

The fine aggregate used was from a single lot of natural siliceous sand from Long Island having a F.M. of 2.60. The coarse aggregate was Connecticut trap rock of 1½-in. maximum size and having a F.M. of 7.0. The characteristics of the aggregate are shown in Table 2.

TABLE 2—PHYSICAL CHARACTERISTICS OF AGGREGATES

Type	Bulk Sp. Grav.	Absorp. per cent	Soundness Loss-% (a)	Strength Ratio	Abrasion Loss-%(b)	Spec. Heat
Stone	2.93	1.3	0.5	....	9.6	0.208
Sand	2.62	0.5	2.5	109	....	....

(a) $MgSO_4$ 5 cycles.
(b) Los Angeles Machine.

Concrete

(a) *Mixtures and rounds.* Mixtures were designed with each of the admixtures to produce nominal cement factors of 4.5, 5.25 and 6.0 bags per cu. yd. of concrete with a nominal slump of 3 in. The W/C and sand-aggregate ratio were permitted to fluctuate as affected by the admixture. The properties of the mixtures used are given in Table 3.

In this initial phase of the work, one round of 4.5- and 6.0- bags cement-factor concrete and three rounds of the 5.25-bag cement-factor concrete were made. In each round, 3.3-cu. ft. batches of concrete were made for each of nine conditions of concrete.

Mixing was accomplished in a 3.5-cu. ft. Smith tilting drum mixer. The batches were mixed for two minutes, allowed to rest for three minutes followed by a one minute re-mixing. Timing of the mixing started when all materials including water had entered drum.

(b) *Specimens.* The specimens prepared from each 3.3-cu. ft. batch were; four 6-in. by 12-in. cylinders, one 6-in. by 6-in. by 30-in. column, one 6-in. by 6-in. by 30-in. beam, three 3½-in. by 4½-in. by 16-in. beams, two 6-in. by 6-in. by 8-in. prisms (for horizontal and vertical pull-out tests), and specimens for air content and bleeding. All specimens were consolidated in their molds by vibration, except for slump and air content tests. External vibration at 3600 r.p.m., was used except that internal vibration, at 7000 r.p.m., was used for the column specimens.

The specimens were cured and tested* as follows:

Cylinders. Stripped at 24 hr. and cured in fog to time of test. Tested at 10 and 30 days.

Columns and beams (large). Stripped at 48 hr. and cured in fog for 16 additional days, then stored in laboratory air pending shipment to Treat Island, Maine for installation on the tidal exposure rack in October 1944.

*The deviation from standard times of stripping, curing and testing was necessitated by the working schedule of the laboratory.

TABLE 3—CONCRETE MIXTURE DATA

Admixture	Cement Factor bags per cu. yd.		W/C g.p.b.	Sand % by Vol.	Unit Weight lb. per cu. ft.	
	Theoret.*	Actual			Theoret.*	Actual
P†	4.5	4.36	8.75	46	154.6	149.5
Q	4.7	4.35	7.40	40	158.0	145.2
R	4.7	4.36	7.40	40	158.0	145.7
S	4.7	4.47	7.40	40	158.0	149.7
T	4.7	4.54	7.45	40	157.9	152.5
U	4.7	4.30	6.75	37	160.1	145.0
X	4.7	4.37	7.50	40	157.8	146.0
Y	4.5	4.41	8.60	46	155.0	151.9
Z	4.7	4.49	6.70	37	160.3	152.8
P†	5.25	5.17	7.25	44.5	155.9	153.4
Q	5.50	5.16	6.25	39	158.7	148.4
R	5.50	5.15	6.10	37	159.4	148.2
S	5.50	5.10	6.10	37	159.4	147.1
T	5.50	5.29	6.35	39	158.4	152.0
U	5.50	5.09	5.85	36	160.3	147.5
X	5.50	5.29	6.30	39	158.5	152.4
Y	5.25	5.15	7.15	44.5	156.2	153.3
Z	5.50	5.24	5.85	36	160.3	152.5
P†	6.0	5.90	6.25	43.5	156.6	153.9
Q	6.2	5.84	5.60	38.5	158.8	149.1
R	6.2	5.82	5.60	38.5	158.8	148.4
S	6.2	5.83	5.60	38.5	158.8	148.6
T	6.2	5.98	5.70	38.5	158.5	152.8
U	6.2	5.85	5.25	35.0	160.5	150.9
X	6.2	5.87	5.60	38.5	158.8	149.9
Y	6.0	5.89	6.20	43.5	156.7	153.8
Z	6.2	5.85	5.20	35.0	160.7	151.0

*Theoretical cement factor and theoretical unit weight are based on calculations of the concrete mixture in an air-free condition.
†P = Concrete without admixture.

Beams (small). Stripped at 48 hr., then cured in fog for 14 days (for laboratory freezing and thawing) or until time of test (for flexural strength).

Prisms (pull-out). Undisturbed in mold and protected against vibrations for 96 hr. (exposed surface coated with membrane compound), then stripped and cured for additional 22 days in fog followed by seven days in laboratory air. Tested at age of 33 days.

Discussion of results—plastic concrete

The characteristics of the unhardened mixtures in this phase of the program are given in Tables 3 and 4. These characteristics are discussed briefly below:

(a) *Water demand.* The data indicate the large reduction in W/C made possible by use of the air-entraining admixtures with the cement factors, and aggregate type and size used. It was noted that due probably to higher air content, the reduction in W/C was greater with the addition of all air-entraining admixtures at the mixer than was practicable with

TABLE 4—CHARACTERISTICS OF PLASTIC CONCRETE MIXTURES

Admixture	Air Content of Mortar—% A.S.T.M. C185-44T	Air Content of Concrete—%			R*			Slump, Inches			Bleeding, per cent		
		Cement Factor[a]			Cement Factor[a]			Cement Factor[a]			Cement Factor[a]		
		4.5	5.25 b	6.0	4.5	5.25 b	6.0	4.5	5.25 b	6.0	4.5	5.25 b	6.0
P[d]	8.2	3.3	1.6	1.7	2.48	5.13	4.82	3	2.6	2.75	15.6	12.8	8.7
Q	17.4	8.1	6.5	6.1	2.15	2.68	2.85	4	3.25	3.50	8.0	7.0	5.6
R	19.2	7.8	6.5	6.5	2.46	2.95	2.95	4	3.1	2.75	8.4	6.7	4.2
S	19.1	5.2	7.8	6.4	3.67	2.45	2.98	2.50	3.25	3	3.2	3.2	2.9
T	17.0	3.4	4.0	3.6	5.00	3.86	4.72	2	3.0	2.75	8.7	7.9	5.6
U	17.6	9.4	8.0	6.0	1.87	2.20	2.93	3	3.2	2.50	5.9	4.6	2.9
X	12.3	7.5	3.9	5.6	1.64	3.15	2.20	5	2.7	2.75	11.2	8.9	4.8
Y	c	2.0	1.9	1.8	6.40	6.16	6.30	3.25	2.9	2.75	12.7	11.6	10.3
Z	20.2	4.7	4.9	6.0	4.30	4.12	3.37	2	2.75	3	6.1	6.7	2.4

Note: Determinations of air content of mortars run in duplicate.
R* = Ratio of Air in Mortar to Air in Concrete.
a : Cement factors are the designed nominal cement factors and are therefore approximate.
b : Values shown for C.F. = 5.25 bags are averages of 3 tests, except in the case of S, where they are averages of 2 tests.
c : C.F. = 4.5, 12.8%; C.F. = 5.25, 11.7%; C.F. = 6.0, 11.5%, due to the addition of three different quantities of admixture.
d : P = Concrete without admixture.

similiar mixtures containing neutralized Vinsol resin interground at the mill (65 and 82) even though, in the case of neutralized Vinsol resin, comparable quantities of the resin were used. It is to be noted, too, that practically no reduction in W/C resulted from the non-air-entraining admixture Y.

(b) *Consistency* (*slump*). The basic mixtures were designed to develop a slump of 3 in. The actual slumps obtained are shown in Table 4.

(c) *Bleeding* (*water-gain*). The large reduction in bleeding* developed by the mixture containing air-entraining admixtures is evident in Table 4. The largest reduction was obtained with admixture S. In general, the air-entraining admixtures reduced the bleeding of the plastic mixtures by at least 40 per cent of that resulting from the use of plain cement.

(d) *Air Content.* The air entrained in standard mortar (A.S.T.M. C 185-44T) and in the concrete mixtures, and the ratios between air-in-mortar and air-in-concrete are shown in Table 4. The particular value of the data is in the relative amounts of air entrained by the various materials in the several types of mixtures. Of value, too, is the record of the highly variable ratio of air-in-mortar to air-in-concrete as it reflects on the ability of A.S.T.M. Method C 185 to predict from air-in-mortar values the amount of air which will be entrained in concrete. The ratios are particularly divergent from the value of 2.54 determined as the average in previous work, in the cases of plain cement and admixtures, T, U, X, Y, and Z. It appears from these data that the amount of air which will be entrained in a given concrete mixture with a given amount of an air-entraining admixture can be determined accurately only by tests using the concrete and admixture involved. *It should be noted that the excessively high air contents of the concrete made with several of the admixtures were not corrected by reducing the amount of the admixture in the mixture because it was desired to use the admixtures in the amounts recommended by the manufacturer or common in previous practice.*

Discussion of results—hardened concrete

(a) *Compressive strength.* The influence of the various admixtures in the amounts used on the compressive strength of concrete containing three different cement factors is shown in Table 5. Admixture Z shows the most beneficial effect on strength with all cement factors and at both ages; as might be expected from the large reduction in water demand resulting from its use. Due, probably to the presence of accelerators, admixtures S and U show a beneficial effect of strength under all conditions of cement factor and age, with S showing the most beneficial result with the low cement factor, where such effect is most needed. None of the air-entraining admixtures shows a materially detrimental effect on

*Determined using method similar to that described in the "Handbook for Concrete and Cement" published by this office; Method CCL-CON-7, p 127.

TABLE 5—RELATIVE COMPRESSIVE AND FLEXURAL STRENGTHS OF CONCRETE

Admixture	Strength—Per Cent								
	Age: 10 days			Age: 30 days					
	C.F.* = 4.5	C.F. = 5.25	C.F. = 6.0	C.F. = 4.5		C.F. = 5.25		C.F. = 6.0	
	Comp.	Comp.	Comp.	Comp.	Flex.	Comp.	Flex.	Comp.	Flex.
P†	100 (2210 psi)	100 (3030 psi)	100 (3720 psi)	100 (2985 psi)	100 (510 psi)	100 (4320 psi)	100 (635 psi)	100 (5420 psi)	100 (730 psi)
Q	99	99	104	95	101	109	97	99	88
R	90	101	92	95	100	105	100	90	100
S	132	117	105	129	115	109	107	103	102
T	86	111	101	107	108	108	72	101	94
U	101	115	112	108	97	115	96	105	101
X	95	127	107	96	108	116	116	103	97
Y	85	99	96	76	67	89	72	81	69
Z	143	152	132	149	130	131	121	125	111

*C.F. = Nominal cement factors in bags per cu. yd.
†P = Concrete without admixture.

strength under any of the conditions of age and cement factor despite rather high air contents.

(b) *Flexural strength.* The influence of the various admixtures on the flexural strength of concrete, containing three cement factors, is shown in Table 5. The results shown are the values obtained on the 3½-in. by 4½-in. by 16-in. beams made and tested in parallel with the durability test specimens, but cured for their entire age in water at 70 F. The data indicate that none of the air-entraining admixtures exerts an appreciably adverse influence on the flexural strength of concrete in spite of rather high air contents.

(c) *Modulus of elasticity.* The modulus of elasticity of the concrete was determined dynamically at an age of 16 days just prior to the freezing and thawing test. The values obtained are given in Table 6 as general information.

TABLE 6—AVERAGE DYNAMIC MODULI OF ELASTICITY

Age: 16 days

Admixture	Nominal Cement Factor b. per c.y.		
	4.50	5.25	6.00
	Dynamic E—psi x 10^6		
P	5.43	6.26	6.35
Q	5.05	5.66	5.75
R	5.15	5.73	5.68
S	5.78	5.49	5.80
T	5.80	5.98	5.98
U	4.95	5.67	5.86
X	5.27	6.24	5.98
Y	4.17	5.12	4.62
Z	6.41	6.27	6.16

(d) *Bond-to-steel.* The tests of bond of concrete, containing the various admixtures, to reinforcing steel were performed using ¾-in. high yield-point HI-BOND bars. However, these data were consolidated with more comprehensive data from subsequent tests and will be discussed later in this paper.

(e) *Durability.* (1) *Field.* Column and beam specimens were installed on the exposure rack at Treat Island in October 1944, but the deterioration of specimens has not progressed to the point permitting any comment or conclusions at this time.

(2) *Laboratory freezing and thawing.* The influence of the various admixtures on the resistance of concrete to rapid freezing and thawing (84) is summarized in Table 7 in terms of durability factors calculated as in Fig. 1. The saturated specimens immersed in water were subjected to reversals in temperature from 42 ± 2 F. to 0 ± 2 F. eleven times in 24 hr., commencing at an age of 16 days.

$$DFE = \frac{PN}{M}$$

where:

DFE = Durability factor in per cent of dynamic E at zero cycles.
P = Relative E of 50 per cent or greater at time of test based on E at zero cycles.
N = Number of cycles at which P reaches 50 cent *or* the ultimate number of cycles of the test prior to completion of test.
M = Ultimate number of cycles of the test.

Fig. 1—Calculation of relative durability factor—based on relative dynamic modulus of elasticity

The data in Table 7 indicate that at 215 cycles of quick freezing and thawing all of the concrete containing air-entraining admixtures is vastly more durable than concrete made with plain cement or with the non-air-entraining admixture Y. The concrete containing admixture R with each of the cement factors was at least slightly more durable than concrete containing any of the other admixtures, with admixture S being only very slightly less durable.

TABLE 7—INFLUENCE OF ADMIXTURES ON DURABILITY OF CONCRETE AS TESTED IN LABORATORY QUICK FREEZER

Durability Factor DFE at 215 cycles

	Nominal Cement Factors in bags per cu. yd.		
Admixture	4.50	5.25	6.00
P*	31	7	25
Q	82	86	86
R	85	88	89
S	84	87	86
T	77	81	83
U	74	81	88
X	84	82	86
Y	18	25	13
Z	76	81	88

*P = Concrete without admixture.

The history of the decrease in dynamic E of the specimens is shown in Fig. 2 to a maximum of 360 cycles of freezing and thawing. The order of decreasing durability is not changed appreciably from that which existed at 215 cycles, but the degree of deterioration is considerably greater, particularly in the case of admixture Z at 5.25 bags per cubic yard, and admixture U at 4.50 bags per cubic yard.

A check on the reliability of the durability factor based on decrement is dynamic E is afforded by the ratio of flexural strength of the frozen and thawed specimens at the ultimate number of cycles of the test, to the flexural strength of similar unfrozen specimens at equal age. The ratio is expressed as the D.F.M., or relative modulus of rupture. The relation-

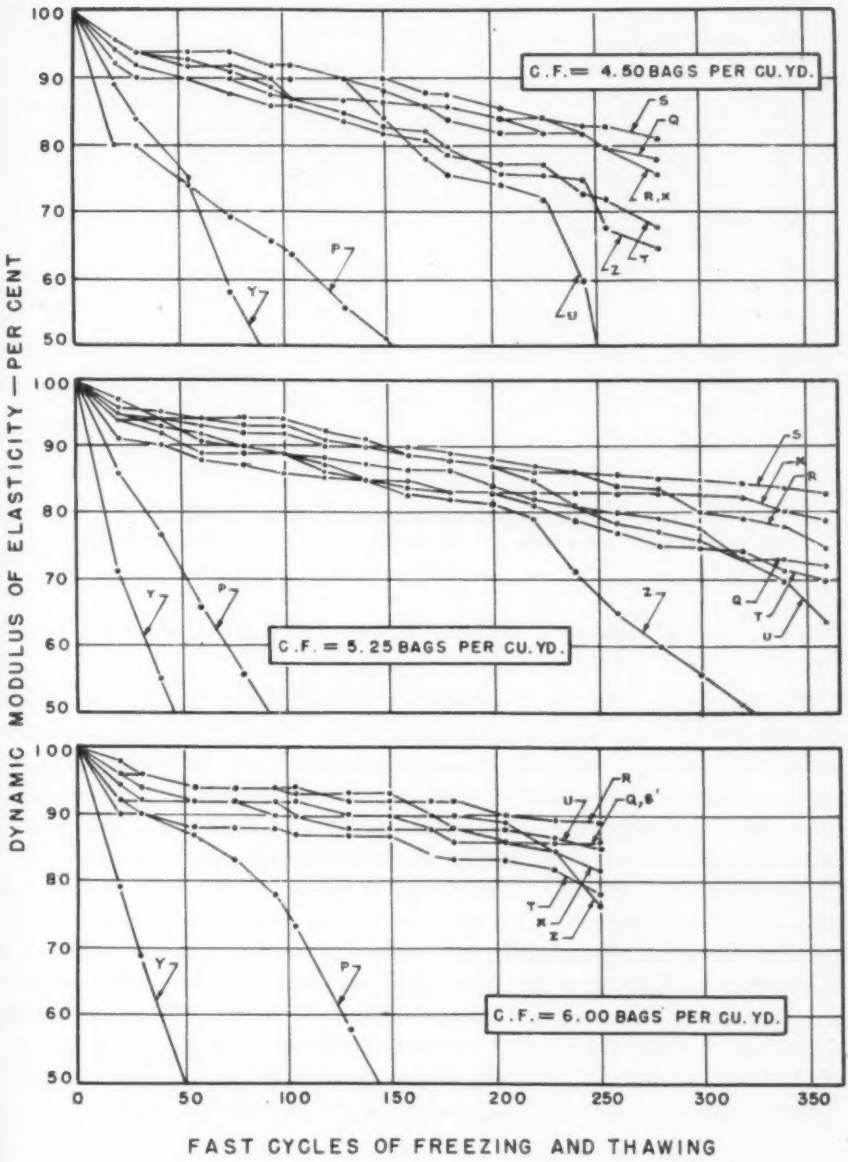

Fig. 2—Influence of type of admixture on resistance of concrete to accelerated laboratory freezing and thawing in terms of dynamic "E" in per cent

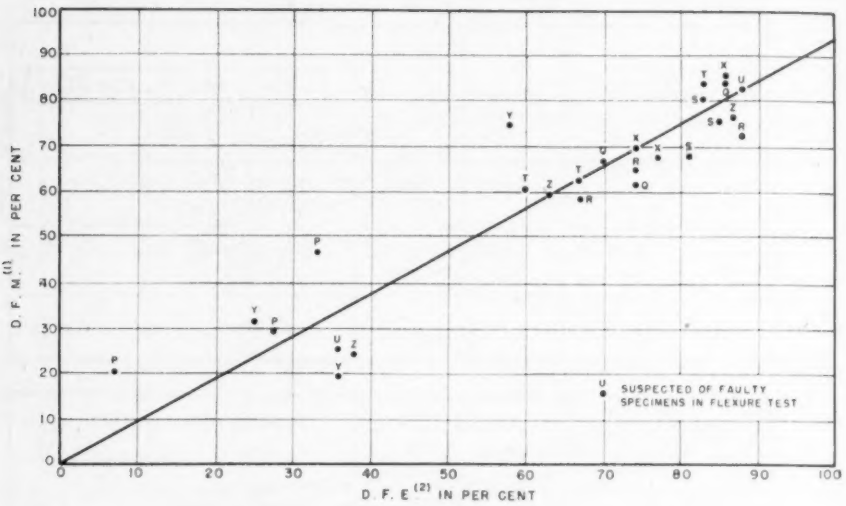

Fig. 3—Relationship between durability factors E and M
(1) Relative modulus of rupture M. (2) Durability factor based on decrement in dynamic E.

ship between D.F.E. and D.F.M. is reasonably good as indicated in Fig. 3.

The foregoing data were considered to be inadequate for final conclusions to be drawn but when considered in conjunction with the work previously accomplished, they were considered to be an adequate basis for the drafting of a tentative specification and methods for evaluating admixtures for concrete (77). The tentative specification was based upon the performance of an admixture with two cements meeting Federal Specification SS-C-206a but differing in reaction with certain of the admixtures. It was a further purpose of the specification to place admixtures in a definite category as a material for use in concrete subject to acceptance testing as are the other materials for concrete: cement and aggregate. The action appeared desirable in view of the beneficial effects on the performance of concrete inherent in the use of certain admixtures providing that protection was afforded against uncertainties of performance or sacrifice in rate and degree of strength gain.

SECOND PHASE

The previous work demonstrated the excellent effect of air-entraining admixtures on the durability, workability, and coherence of concrete mixtures, but additional data were required on the effect of various types of admixtures on these and other properties of concrete.

TABLE 7A—CHEMICAL ANALYSES OF CEMENTS "A" AND "C"

	Cement A	*Cement C*
Oxides:		
SiO_2	22.20	21.15
Al_2O_3	4.28	5.08
Fe_2O_3	3.42	4.05
CaO	64.96	63.42
MgO	1.26	2.09
SO_3	1.30	1.80
Ign. Loss	1.27	0.88
Ins. Residue	0.08	0.14
Free CaO	1.02	0.41
P_2O_5	0.150	0.229
Mn_2O_3	0.051	0.055
Na_2O	0.167	0.130
K_2O	0.463	0.727
H_2O Sol. Na_2O	0.031	0.083
H_2O Sol. K_2O	0.164	0.443
Compounds:		
C_3S	58.3	52.3
C_2S	19.8	21.3
C_3A	5.6	6.6
C_4AF	10.4	12.3
$CaSO_4$	2.2	3.1
Ratios:		
CaO Saturation	69.26	67.64
Colony's Ratio	2.68	2.65
Al_2O_3/Fe_2O_3	1.25	1.25
SiO_2/R_2O_3	2.88	2.32
CaO/SiO_2	2.93	3.00

Materials

For this purpose, an extended program was set up based on the tentative specification referred to above, using the following materials:

(a) *Cements.* Commercial SS-C-206a cements from two mills.*

(b) *Aggregate.* Natural siliceous sand (F.M. 2.60) and Connecticut trap rock (1½-in. max.) as used in the preliminary programs.

(c) *Admixtures*(†). The following admixtures were added at the mixer at the time of mixing in the form and proportions previously described: Q, R, U, V, and Z. The symbol P was again used to designate concrete made without admixture.

Mixtures, mixing and specimens

Except where special factors were to be determined requiring a deviation therefrom; the mixtures were designed to have nominal actual cement factors of 4.5 and 6.0 bags per cu. yd. with W/C and sand contents varying as made practicable by the characteristics of the admixtures to develop a nominal slump of 2½ in. Preliminary and small batches (0.8 cu. ft.) were mixed in a 1-cu. ft. tilting-drum mixer. The preponderant

*See Table 7A for chemical analyses of these cements.

†It is emphasized that the commercial admixtures used were of the composition and quality commercially available in August, 1944.

number of batches were 2.75 cu. ft. in volume and were mixed in a 3.5-cu. ft. tilting-drum mixer. Following the addition of all materials including water, all batches were mixed for 2 min. followed by a 3-min. resting period and a 1-min. re-mixing period. All batches were discharged into a metal pan or boat and turned over three times with a shovel and then covered with damp burlap to minimize loss of moisture during the molding of specimens. The molding of all specimens was accomplished by rigid adherence to standard practice and each operation was performed insofar as practicable by the same person throughout the program. All conditions and variables tested were repeated on at least three different days. The principal types of specimens were removed from their molds and cured, as follows, with deviations therefrom explained in the succeeding text as they occurred:

(a) *Cylinders.* Stripped at 24 hr., cured in fog at 70 ± 2 F. until time of test.

(b) *Beams (3½ by 4½ by 16 in.) and bars (2 by 2 by 11 in.)* Exposed surface coated with membrane compound 2 hr. after molding. Stripped at 48 hr., and cured in fog at 70 ± 2 F. until time of test.

(c) *Prisms (pull-out).* Exposed surface coated with membrane compound 2 hr. after molding. Stripped at 96 hr. and cured in fog at 70 ± 2 F. until time of test at 28 days.

Discussion of results—plastic concrete

The effects of the various admixtures on the principal properties of the plastic and hardened concrete duplicated with two cements are discussed in the following paragraphs. *No attempt was made to regulate the air content of the mixtures to an optimum amount by varying the amounts of the admixtures added, although such regulation is possible and most desirable in obtaining the optimum benefits from air entrainment.* Deviation from the standard procedures described above are explained wherever they occur.

(a) *Water demand.* Using mixtures having nominal actual cement factors of 4.5 and 6.0 bags per cu. yd., the influence of the admixtures on the W/C necessary to develop a nominal slump of 2.5 in., was determined in 0.8-cu. ft. batches of concrete. The water-cement ratios so determined were then used in the remainder of the program in 0.8 and 2.75 cu. ft. batches except where otherwise especially noted. The results were similar with both cements and are tabulated in Table 8 together with the sand-aggregate ratios used.

(b) *Cement demand.* The effect of the admixtures on the cement content of concrete mixtures having water-cement ratios of 5.5 and 6.5 gal. per bag and in which the slump was maintained at 2.5 in. is shown in Table 9. These mixtures were made in 0.8-cu. ft. batches and the values were verified by check tests on different days.

TABLE 8—EFFECT OF ADMIXTURES ON WATER-CEMENT RATIO

Admix.	Cement Factor = 4.5 b/cu.yd.			Cement Factor = 6.0 b/cu.yd.		
	S/A % by Vol.	W/C g.p.b.	Reduction g.p.b.	S/A % by Vol.	W/C g.p.b.	Reduction g.p.b.
P*	46	8.60	0	43.5	6.25	0
Q	41	7.35	1.25	37.5	5.60	0.65
R	41	7.40	1.2	37.5	5.65	0.60
U	39.5	6.90	1.7	37.5	5.30	0.95
V	41	7.35	1.25	37.5	5.60	0.65
Z	44	7.75	0.85	40.5	5.75	0.50

*Concrete without admixture.

TABLE 9—EFFECT OF ADMIXTURES ON CEMENT DEMAND

Admix.	W/C = 5.5 g.p.b.			W/C = 6.5 g.p.b.		
	S/A % by Vol.	C.F.*	Reduction b/cu.yd.	S/A % by Vol.	C.F.*	Reduction b/cu.yd.
P†	42	6.80	0	44	5.40	0
Q	38	6.12	0.68	40	4.86	0.54
R	38	6.16	0.64	40	4.90	0.50
U	37	5.55	1.25	38.5	4.25	1.15
V	38	6.15	0.65	40	4.90	0.50
Z	39	6.17	0.63	40.5	4.80	0.60

*Actual cement factor in bags per cu. yd.
†Concrete without admixture.

TABLE 10—EFFECT OF ADMIXTURES ON SLUMP

Admix.	Theo. Cement Factor = 4.6 b/cu.yd. W/C = 8.60 g.p.b. S/A = 46 per cent by vol.			Theo. Cement Factor = 6.1 b/cu.yd. W/C = 6.25 g.p.b. S/A = 43.5 per cent by vol.		
	Cement A Slump, in.	Cement C Slump, in.	Aver. Increase Slump, in.	Cement A Slump, in.	Cement C Slump, in.	Aver. Increase Slump, in.
P*	2.25	2.25	0	2.50	2.50	0
Q	6.25	4.00	3	4.75	5.00	2.5
R	5.50	4.25	3	5.75	4.75	3
U	8.00	7.25	5.5	7.00	6.50	4
V	6.75	4.00	3	5.75	5.00	3
Z	5.25	5.00	3	6.75	5.75	4

*Concrete without admixture.

(c) *Consistency.* The effect of admixtures on consistency (slump) of concrete mixtures, when the W/C, sand-aggregate ratio and cement content used with plain cement are held constant is shown in Table 10 for relatively lean and rich mixtures.

(d) *Bleeding.* The effect of admixtures on bleeding of concrete mixtures, in which the actual cement factors and slump remain constant and the W/C and S/A were as shown in Table 8 is shown in Table 11.

TABLE 11—EFFECT OF ADMIXTURES ON BLEEDING

Admix.	Cement Factor = 4.5 b/cu.yd.		Cement Factor = 6.0 b/cu.yd.	
	Cement A Bleeding —%	Cement C Bleeding —%	Cement A Bleeding —%	Cement C Bleeding —%
P	100 (14.8)	100 (14.1)	100 (9.7)	100 (7.4)
Q	52	49	42	34
R	59	56	49	49
U	25	25	19	15
V	42	33	36	30
Z	55	57	25	20

Note: Figures in parentheses are average values of bleeding by concrete containing no admixture.

(e) *Air content.* The effect of admixtures on the air content of concrete mixed for a total period of 3 min. as previously described but with differing conditions of design is shown in Table 12. These data indicate the considerable influence of type of mixture, type of cement, and amount of admixture on air content. These effects are subject to control by regulating the amount of admixture used.

(f) *Effect of mixing time on air content.* The effect of time of mixing on air content with various admixtures was studied concurrently with and as a part of this program. Even though the conditions of the test were somewhat different than those which have been described, it is felt that the data should be inserted in this position in the paper.

The influence of cement itself on air entrainment in the presence of an admixture (which did not depend on a chemical reaction with cement to become active) was tested by mixing large (approx. 2.75-cu. ft.) batches of Ottawa sand, neutralized Vinsol resin, and water in a 3.5-cu. ft. tilting mixer. The air content of the sand-water mixture was determined at frequent intervals and mixing was continued until the air content began to diminish from the maximum. A similar test was run in which calcium oxide was added to the water to determine whether possible precipitation of calcium salts might affect the air entrainment. The foregoing tests on cement-free sand-water mixtures were duplicated using a simulated concrete mixture with quartz pebbles ½-in. in size as coarse aggregate to determine the influence of such pebbles on air entrainment in the absence of cement. A third group of tests was conducted in which Cements A and C were added to the simulated concrete mixture. A fourth series of tests was made using Cements A, B, C, and D in regular concrete mixtures. In this fourth series, each of the cements

TABLE 12—EFFECT OF ADMIXTURES ON AIR CONTENT OF CONCRETE MIXTURES WITH VARIOUS CONSTANT BASES ON DESIGN

a

Constant Actual Cement Factor * b/cu. yd.	Cement	Air Content, per cent Admixture					
		P	Q	R	U	V	Z
4.5	A	1.6	6.2	5.2	10.7	6.0	5.3
4.5	C	2.3	7.6	6.5	10.7	6.6	5.5
6.0	A	1.5	4.7	4.5	7.7	4.4	4.6
6.0	C	2.1	5.6	5.7	9.7	4.4	4.7

b

Constant W/C & Slump†		Cement	Air Content, per cent Admixture					
W/C g.p.b.	Slump, in.		P	Q	R	U	V	Z
5.5	2.5	A	1.4	4.2	3.6	6.0	3.8	3.1
6.5	2.5	A	1.3	3.9	3.5	6.4	3.3	4.1
5.5	2.5	C	1.3	5.4	5.3	8.2	5.9	4.0
6.5	2.5	C	1.4	4.4	4.8	8.7	5.1	4.9

c

Constant Theo. Cement Factor, W/C, and S/A‡			Cement	Air Content, per cent Admixture					
Theo. C.F. b/cu. yd.	W/C g.p.b.	S/A % by Vol.		P	Q	R	U	V	Z
4.6	8.6	46	A	1.6	6.2	5.7	8.7	6.7	4.9
4.6	8.6	46	C	2.3	7.0	6.9	8.3	7.0	4.0
6.1	6.25	43.5	A	1.5	5.5	4.7	8.1	6.3	5.1
6.1	6.25	43.5	C	2.1	7.3	6.7	10.1	7.8	5.9

*See Table 8. †See Table 9. ‡See Table 10

was tested, (a) with no admixture, (b) with interground flake Vinsol resin, (c) with interground neutralized Vinsol resin, and (d) with a solution of neutralized Vinsol resin added to the batch at the mixer. A fifth series of tests was made in which Cements A and C were used in regular concrete mixtures with and without several other types of air-entraining admixtures. Samples were extracted from the mixer for air-content determination at regular intervals and mixing was continued until the air content diminished from the maximum. The mortar and simulated concrete batches were made with silica sand and quartz pebbles to minimize any possible opportunity for grinding action to affect the entrainment of air. The results of these tests are presented in Fig. 4 and 5.

The data in Fig. 4a indicate that the presence of $Ca\ (OH)_2$ in solution increases the tendency toward entrainment of air, but the influence of

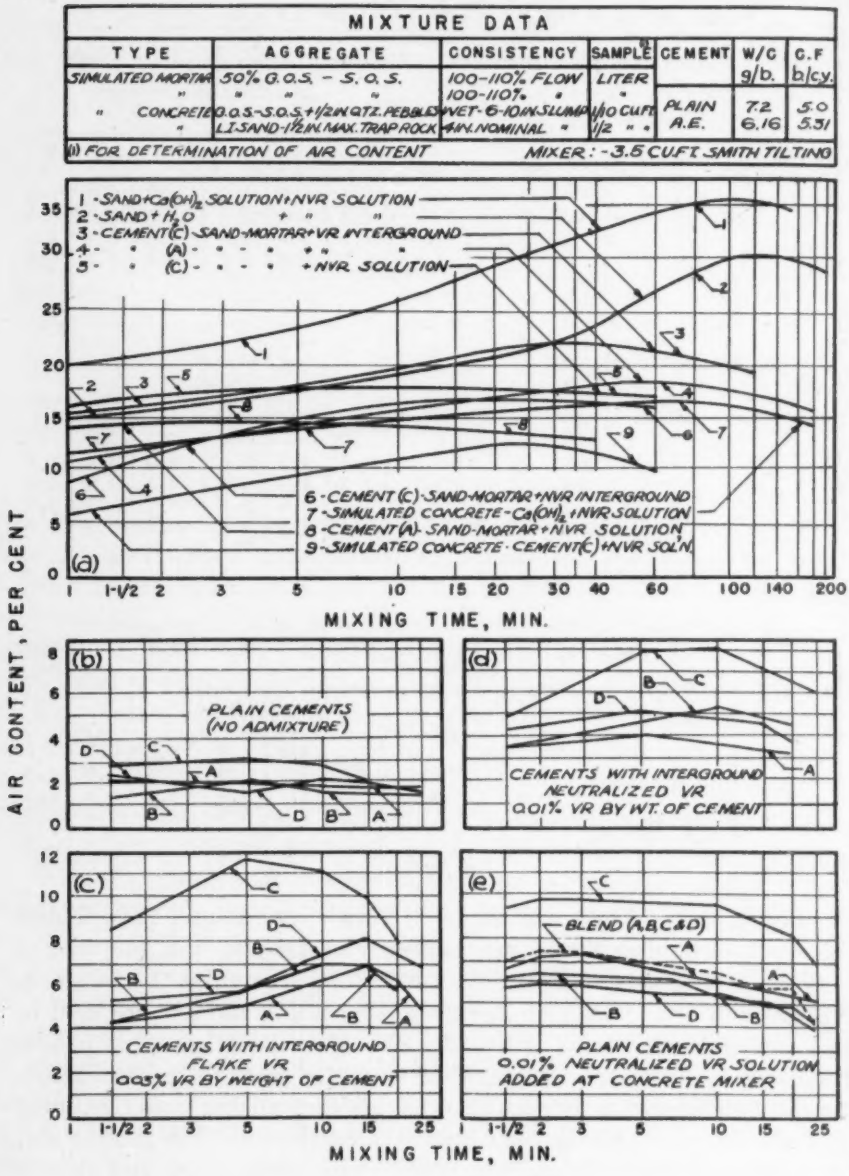

Fig. 4—Relation of air content to length of mixing time

the cement itself on air entrainment is negative, that is; the amount of air entrained and the duration of additional air entrainment are reduced appreciably by the presence of cement. This is in accord with the findings of others (65) that air entrainment is particularly a function of the amount and grading of the fine aggregate. As the fineness of the sand or the ratio of cement to sand increases, the tendency toward air entrainment is reduced. The simulated concrete batches entrained considerable air and were coherent and plastic mixtures despite the complete absence of cement.

The data in Fig. 4b indicate the amount of air entrained in concrete without admixture, containing four different brands of cement and the rate and degree to which the air is entrained. There is no evidence of increase in air content with continued mixing although there is some difference in the total amount of air entrained by the four cements used.

Fig. 4c shows the effect of time of mixing on the air entrainment of the concrete mixtures cited when the four basic plain cements are interground with 0.03 per cent flake Vinsol resin. In this case, air entrainment increases for an appreciable time before reaching a maximum and commencing to recede. This tendency is due, probably, to the continued reaction between the unneutralized Vinsol resin and the soluble alkalies present in the cements. Each of the four cements reacts differently under the influence of equal amounts of Vinsol resin.

Fig. 4d shows the effect of time of mixing on the air entrainment when the four basic plain cements are interground with 0.01 per cent neutralized Vinsol resin. Air entrainment continues with mixing time but the peaks are reached at an earlier period and the differences in air content are not as great as in Fig. 4c. The delay in air entrainment is due, probably, in this case to adsorption of the interground neutralized Vinsol resin by the particles of cement.

Fig. 4e shows the effect of time of mixing on the air entrainment when 0.01 per cent neutralized Vinsol resin is added at the mixer to similar concrete batches containing the four basic plain cements. In this case, the maximum air entrainment is reached early and recedes thereafter, and the differences between the cements are minimized. This action is due, probably, to the immediate and complete availability of all the admixture.

Fig. 5 shows the effect of time of mixing on the air entrainment when the recommended quantities of several of the admixtures are added at the mixer with two of the basic plain cements. The rate of air entrainment and the recession in air content is generally similar to that shown in Fig. 4e with neutralized Vinsol resin when added at the mixer.

The recession in air content in all cases where appreciable air is entrained is due, probably, to final exhaustion of the sudsing property

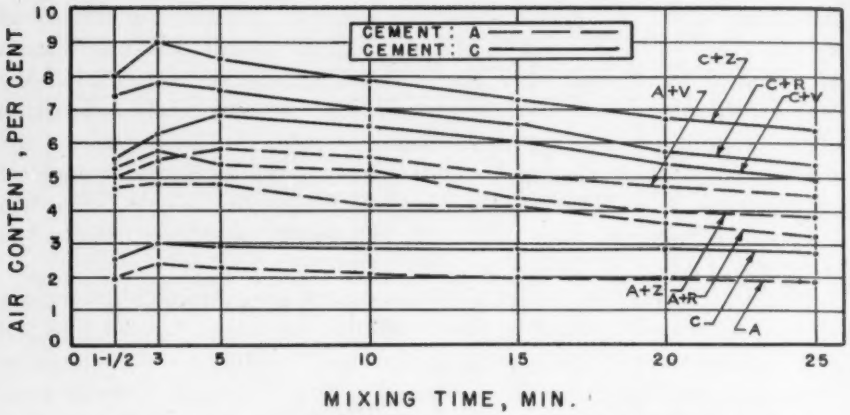

Fig. 5—Relation of mixing time to air content of concrete made with and without admixtures

of the admixture and escape or bursting of some of the air bubbles. In mixtures containing readily grindable aggregates, recession in air content may be accentuated by increased fineness of cement and aggregate due to the grinding action.

The data from the tests described above are important particularly as they reflect the influence of *time* of mixing on the air content of concrete. The particular amounts of air entrained are less important, because these *amounts* are regulable by adjusting the amount of the admixture used. Such regulation of amount of air and amount of admixture is most practicable when the admixture is added at the mixer.

It should be in mind, too, that these tests were made in a stationary tilting-drum mixer and the results may or may not have relationship to air entrained by transit mixers.

Discussion of results—hardened concrete

(a) *Compressive strength.* The effect of the admixtures on compressive strength of concrete mixtures made under various conditions is shown in Tables 13, 14, and 15; and in Fig. 6.

Under normal conditions of standard making and curing (70 ± 2 F.) and with the proportions given in Table 8, the admixtures affect the strength of concrete at the ages of 2 and 28 days as shown in Table 13, in terms of strength gained at equal ages with the two cements and no admixture. The effect of the accidental use of double quantities of each of the admixtures is also shown. It is apparent from these data that the large reductions in W/C made possible by addition of the admixtures at the mixer reinforced by the presence of accelerators in some of the admixtures resulted in superior strength, as compared to concrete without admixture, at the early age of two days in all cases except with ad-

TABLE 13—EFFECT OF ADMIXTURES ON COMPRESSIVE STRENGTH

Admixture	Cement A				Cement C			
	Recommended Amount of Admixture		Doubled Amount of Admixture		Recommended Amount of Admixture		Doubled Amount of Admixture	
	2 days	28 days	2 days	28 days	2 days	28 days	2 days	28 days
Cement Factor = 4.5 b./cy. yd.								
	860 psi	2950 psi			1055 psi	3040 psi		
P‡	100%	100%	.....	.....	100%	100%	.....	.....
Q	137%	119%	107%	88%	110%	113%	112%	86%
R	133%	120%	87%	88%	100%	110%	96%	84%
U	119%	106%	85%	84%	93%	114%	95%	94%
V	190%	136%	160%	93%	144%	124%	127%	102%
Z	135%	122%	*	65%*	113%	119%	114%	106%
Cement Factor = 6.0 b./cu. yd.								
	1630 psi	5260 psi			2235 psi	5525 psi		
P‡	100%	100%	.....	.....	100%	100%	.....	.....
Q	102%	100%	107%	83%	109%	96%	97%	84%
R	102%	99%	109%	82%	107%	94%	74%	73%
U	124%	96%	98%	79%	78%	84%	77%	81%
V	159%	150%	140%	87%	115%	105%	102%	86%
Z	109%	102%	*	92%*	107%	106%	88%	84%

*Specimens from one round crumbled on stripping at 24 hr.
‡P = Concrete without admixture.

mixture U combined with Cement C. The improvement in early strength is uniformly greater in the lean than in the rich mixtures; with admixture V developing the most marked improvement.

The effect on strength at the early age is considered to be most important due to its influence on the removal of forms, early use of the structure, and the fact that the thin and exposed sections of structures rarely if ever receive curing comparable with storage in fog at 70 ± 2 F. continuously for 28 days.

The effect of the admixtures on strength is beneficial in all cases with the lean mixture and both cements at the 28-day age. However, the effect is not marked in the rich mixture and is somewhat less than parity in several cases; particularly so with Cement C.

The effect of doubling the quantity of the admixtures is particularly variable at the early age, with admixture Z failing to harden sufficiently to resist the minor stress of form removal at an age of 24 hr., and admixture V maintaining a large improvement in strength except in the rich mixture with Cement C. At 28 days the variability of effect is not great and the strength is generally somewhat less than parity with concrete containing no admixture. It is evident from these tests that admixture Z is the only admixture tested which is adversely affected to a serious extent by the accidental use of a double quantity in a batch of concrete and this effect is noticeable with only one of the two cements used.

When advantage is taken of the workability contributed by the admixtures by reducing the cement content (Table 9) instead of reducing the W/C, the effect on compressive strength at the age of seven days* is as indicated by the data in Table 14.

*No tests made at other ages.

TABLE 14—EFFECT OF REDUCTION IN CEMENT CONTENT WITH ADMIXTURES ON RELATIVE COMPRESSIVE STRENGTH OF CONCRETE AT SEVEN DAYS AGE

	W/C = 5.5 g.p.b. Cement		W/C = 6.5 g.p.b. Cement	
Admix.	A	C	A	C
P*	4180psi. 100%	4395psi. 100%	2440psi. 100%	2550psi. 100%
Q	92	91	92	90
R	90	88	91	89
U	88	84	79	76
V	99	96	111	108
Z	105	101	113	108

*Concrete without admixture.

The influence of low temperature on the rate of early strength gain with and without admixtures, was tested by pre-cooling all ingredients and mixing the concrete at a temperature of 40 ± 2 F. and then storing one-half of the number of specimens in each batch in fog at 70 ± 2 F. and storing the other half of the specimens, sealed in their molds, at 40 ± 2 F. (rel. humidity 80 ± 10 per cent). Companion specimens were then tested in compression at ages of 1, 2, 3, and 7 days. The results are shown in Table 15 and in Fig. 6. The strength of the plain portland-cement mixture is reduced markedly by the lower temperature of mixing and curing and that the reduction in strength and the rate of strength gain for the first seven days is not affected appreciably by any of the admixtures except V. Admixture V exerts a marked beneficial effect on early strength even at a low temperature.

TABLE 15—INFLUENCE OF LOW TEMPERATURE ON STRENGTH DEVELOPMENT OF CONCRETE

Nominal cement factor = 6 bags per cu.yd. Cement A. W/C and S/A as shown in Table 8

Admix.	Strength Ratios — Age in Days							
	Strength at 40 F. as % of strength of plain concrete cured at 70 F. at equal age.				Strength at 40 F as % of strength of plain concrete cured at 40 F.			
	1	2	3	7	1	2	3	7
					89psi.	310psi.	505psi.	1470psi.
P*	18	26	28	45	100	100	100	100
Q	14	18	26	43	78	70	92	96
R	19	23	29	42	105	91	102	93
U	6	13	23	50	32	52	80	111
V	60	55	60	62	285	193	211	138
Z	9	23	34	60	52	91	121	134

*Concrete without admixture.

(b) *Flexural strength.* The effect of admixtures on the flexural strength of concrete mixtures, under standard conditions and with the proportions shown in Table 8, at the age of 28 days is shown in Table 16 in terms of the strength of plain concrete. The effect of a doubled quantity of the admixture is also shown. The data indicate that the strength developed in the presence of the admixtures is approximately equal to that of plain concrete, except that there is a uniform improvement in strength in the concrete containing the admixture V.

(c) *Bond-to-steel.* The effect of admixtures on the bond of concrete* (Table 8) to steel was tested by determining the unit load in lb. per sq. in. of steel embedment at a slip of 3×10^{-4} in. measured at the free

*Proportioned as shown in Table 8 and with air contents as shown in Table 12a.

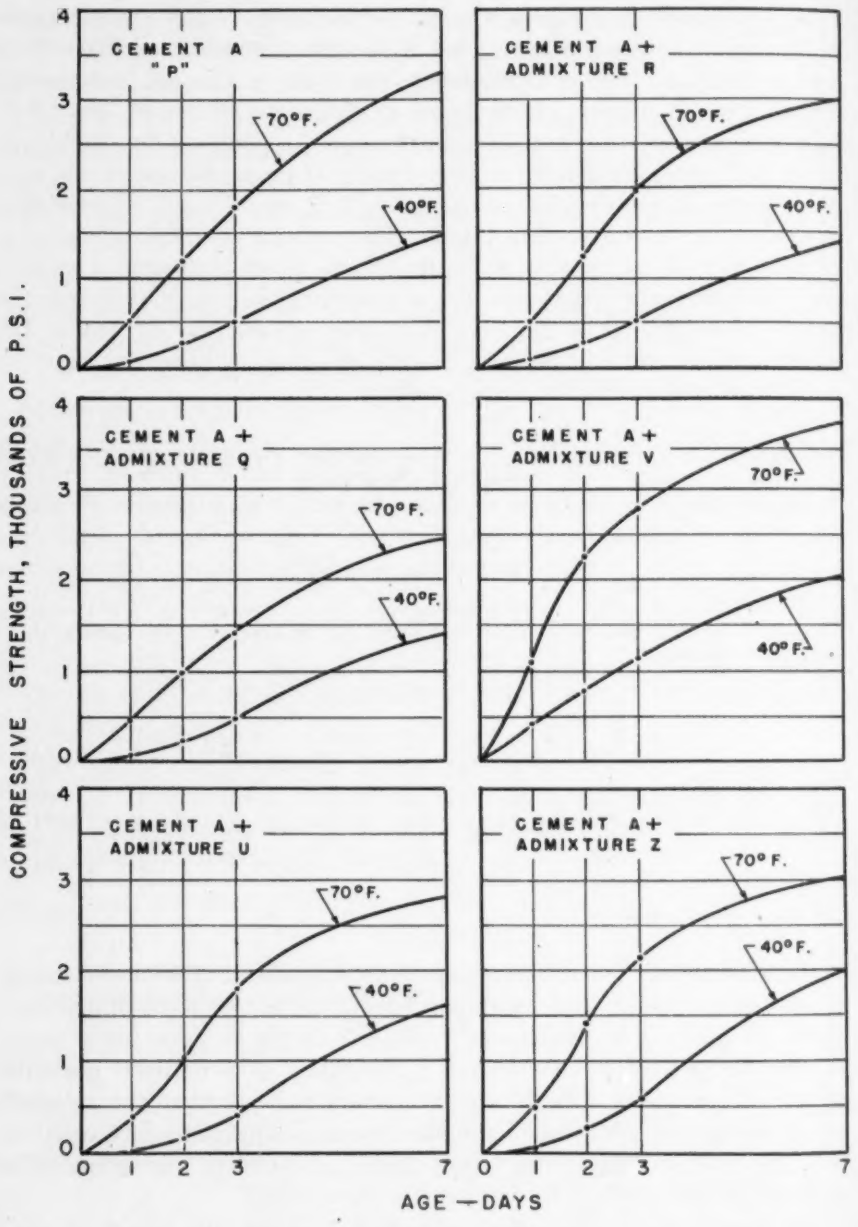

Fig. 6—Influence of temperature on strength development of concrete
Cement factor: 6 bags per cu. yd. Concrete mixed at 40 F., cured at temperatures shown.

TABLE 16—RELATIVE EFFECT OF ADMIXTURES ON FLEXURAL STRENGTH OF CONCRETE

Admix.	Cement A		Cement C	
	Recommended Amt. of Admix.	Doubled Amt. of Admix.	Recommended Amt. of Admix.	Doubled Amt. of Admix.
	Cement Factor = 4.5 bags per cu. yd.			
P[b]	495 psi 100%	—	555 psi 100%	—
Q	105	96	93	91
R	109	96	100	89
U	101	90	100	88
V	119	111	114	109
Z	109	67[a]	98	102
	Cement Factor = 6.0 bags per cu. yd.			
P[b]	705 psi 100%	—	740 psi 100%	—
Q	98	98	98	97
R	104	95	106	90
U	99	87	89	89
V	112	102	112	105
Z	101	73[a]	103	92

(a) Specimens from one round crumbled on stripping at 48 hr.
(b) Concrete without admixture.

end of ¾-in. round, high yield-point, Hi-Bond bars. The bars were cast in a horizontal position in 6-in. by 6-in. by 8-in. prisms and were left completely undisturbed for 96 hr. prior to removal of forms. The pull-out tests were made at an age of 28 days on a total of 454 specimens. The specimens were made in groups of three and repeated on three to seven different days.

The grand average results obtained from all the tests made with nominal cement factors of 4.5 and 6.0 bags per cu. yd. and two cements are shown in Table 17.

TABLE 17—BOND STRENGTH OF CONCRETE TO STEEL

Admix.	Cement A				Cement C			
	C.F. 4.5 bags per cu. yd.		C.F. 6.0 bags per cu. yd.		C.F. 4.5 bags per cu. yd.		C.F. 6.0 bags per cu. yd.	
	psi	%	psi	%	psi	%	psi	%
P*	354	100	412	100	423	100	470	100
Q	270	76	412	100	300	71	473	100
R	230	65	355	86	342	81	496	106
U	265	75	485	118	290	69	541	115
V	540	153	860	209	504	119	1054	224
Z	430	121	582	141	302	71	638	136

*Concrete without admixture.

TABLE 18—EFFECT OF ADMIXTURES ON LENGTH CHANGE IN CONCRETE

Length Change in Per Cent x 10^{-3}

Admix.	Age in Days																			
	Initial Stor. in Fog				Drying at 70 F. — 25 ± 5 R. H.								Resaturation at 70 F.							
	Cement A		Cement C		Cement A				Cement C				Cement A				Cement C			
	7	14	7	14	1	7	14	28	1	7	14	28	1	7	14	28	1	7	14	28
									Cement Factor—4.50 b./cu. yd.											
P*	+5	+6	+4	+7	−10	−31	−50	−63	− 9	−37	−60	−66	+37	+41	+46	+49	+38	+45	+50	+51
Q	+3	+6	+4	+5	−12	−38	−54	−61	−13	−41	−57	−63	+34	+43	+47	+50	+33	+46	+49	+52
R	+4	+6	+4	+6	−10	−31	−48	−58	−11	−36	−53	−61	+35	+43	+47	+49	+35	+46	+48	+51
U	+5	+8	+5	+8	−11	−36	−51	−57	−11	−39	−51	−57	+32	+45	+48	+48	+32	+44	+48	+50
V	+6	+7	+6	+7	−12	−33	−49	−58	−12	−38	−54	−62	+36	+47	+51	+53	+35	+48	+52	+54
Z	+4	+6	+7	+9	−11	−34	−50	−58	−11	−36	−50	−61	+34	+47	+49	+50	+36	+46	+49	+53
									Cement Factor—6.0 b./cu. yd.											
P*	+4	+6	+5	+5	− 8	−29	−39	−48	−10	−33	−50	−56	+36	+41	+45	+46	+38	+44	+49	+55
Q	+5	+5	+6	+7	−11	−37	−46	−53	−11	−34	−46	−49	+36	+44	+49	+51	+33	+42	+46	+50
R	+3	+5	+5	+7	−11	−37	−48	−57	−10	−35	−52	−58	+37	+45	+50	+54	+39	+48	+53	+59
U	+5	+7	+8	+10	−12	−41	−42	−51	−13	−37	−50	−57	+28	+44	+46	+51	+29	+46	+51	+56
V	+8	+8	+8	+9	−14	−41	−50	−59	−10	−32	−46	−53	+35	+48	+51	+54	+33	+47	+50	+53
Z	+8	+9	+7	+8	−11	−38	−46	−56	− 9	−32	−47	−54	+37	+48	+49	+53	+33	+44	+48	+51

*Concrete without admixture.

The deviation in individual results was considerably less than developed in previous tests using bars with diamond-shaped deformations and less careful handling of the specimens at the very early ages, however, the variability in results was sufficiently great to make the general average values of more importance in terms of relativity with plain concrete than in terms of specific strengths in lb. per sq. in. of steel embedment. Since this paper is concerned particularly with the relative effect of the admixtures on bond to steel, no special comment on bond strength as affected by type of bar or other factors will be made.

The data indicate that bond to steel is affected adversely to a serious degree in concrete containing a cement content of 4.5 bags per cu. yd. and admixtures Q, R, or U. With admixture Z, the bond is benefitted when used with Cement A but is relatively poor when used with Cement C. The admixture V has a markedly beneficial effect on bond with both cements.

The data indicate, also, that when used in concrete having a cement content of 6.0 bags per cu. yd., all of the admixtures except R achieve at least parity with plain concrete. Admixture R reduced bond with Cement A and slightly increased bond with Cement C. Admixtures V and Z caused a material increase in bond with both cements; the increase with admixture V being very large, and verified by seven separate runs of three specimens with each cement.

(d) *Volume change.* The effect of admixtures on volume change of concrete was tested in the manners and with the results described below:

(1) *Normal shrinkage and expansion.* The effect of wetting and drying on concrete made with and without the admixtures was tested by molding 2-in. by 2-in. by 11-in. beams from the mixtures referred to in Table 8, except that aggregate larger than ¾ in. was removed by hand picking. The beams were cured in fog (70 ± 2 F.) for 14 days and then stored in air (70 ± 2 F.) at 25 ± 5 per cent R.H., circulated over the specimens at a speed of 15 m.p.h. for 28 days. At the end of this period, the beams were stored in water at 70 ± 2 F. for an additional period of 28 days. The reversals of drying and wetting were continued for three cycles.

The results of the tests indicate that none of the admixtures exerts an appreciable influence on the reaction of the concrete to wetting and drying. Summarized data for the two cements and two cement contents are shown in Table 18 and typical expansion and contraction curves are shown in Fig. 7. Additional data are summarized in Table 19 to show the effect of three repeated cycles of wetting and drying on the length change of concrete made with and without the admixtures and with the blend of four cements as used in the initial phase of the work. The immaterial effect of the admixture on this property of concrete makes further presentation of detailed data of little value to this paper.

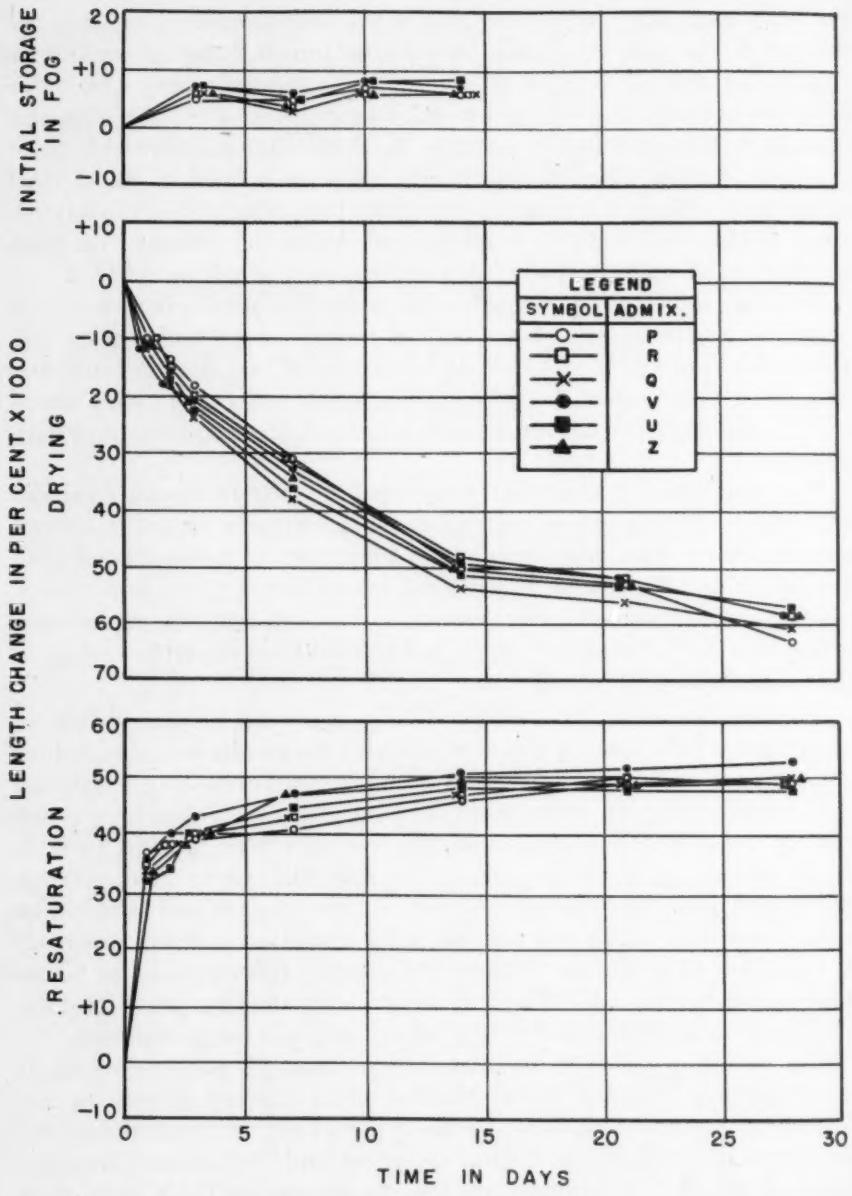

Fig. 7—The effect of wetting and drying on length change of concrete containing various admixtures with cement A

Cement factor: 4.5 bags per cu. yd.

TABLE 19—LENGTH CHANGE OF CONCRETE DUE TO WETTING AND DRYING

Admix.	Cycle	Length Change in Per Cent x 10^{-3}								
		After 14 days Fog Cure at 70 F.			After 28 days drying at 90 F. and 25 ± 5% Rel. Hum.			After 28 days saturation at 70 F.		
		Cement Content b./cu. yd.			Cement Content b./cu. yd.			Cement Content b./cu. yd.		
		4.50	5.25	6.0	4.50	5.25	6.0	4.50	5.25	6.0
P†	1	+ 5	+11	+ 4	−53	−63	−50	+40	+36	+45
	2				−32	−34	−32	+37	+33	+36
	3				−27	−29	−28	+32	+39	+34
Q	1	+ 6	+10	+ 2	−55	−57	−54	+47	+43	+44
	2				−37	−35	−34	+40	+43	+41
	3				−32	−32	−29	+38	+39	+36
R	1	+ 6	+10	+ 3	−53	−54	−50	+41	+41	+39
	2				−33	−33	−33	+36	+40	+40
	3				−28	−28	−38	+34	+35	+44
S	1	+12	+ 6	+ 7	−48	−61	−50	+41	+47	+41
	2				−31	−37	−36	+37	+42	+38
	3				−26	−32	−39	+36	+39	+48
T	1	+ 7	− 2	+ 4	−56	−77	−52	+34	*	+37
	2				−30	*	−34	+35	*	+42
	3				−25	*	−39	+31	*	+45
U	1	+10	+14	+ 5	−52	−52	−49	+33	+42	+47
	2				−32	−32	−43	+34	+39	+35
	3				−23	−29	−32	+34	+35	+43
X	1	+ 6	+14	+ 2	−49	−54	−49	+40	+41	+39
	2				−33	−32	−33	+39	+39	+40
	3				−27	−31	−37	+35	+35	+46
Y	1	No Initial Curing in Fog			−55	−51	−58	+42	+41	+53
	2				−33	−35	−37	+38	+31	+41
	3				−30	−27	−32	+35	+36	+37
Z	1	+ 8	+14	+ 5	−44	−48	−49	+39	+44	+43
	2				−29	−32	−29	+35	+38	+34
	3				−25	−33	−30	+35	+35	+36

*After 28 days drying pin became loose—hence no further length change measurement.
†P = Concrete without admixture.

(2) *Early shrinkage and expansion.* The effect of admixtures on the volume constancy of concrete mixtures in the transition from the plastic to the well-hardened state was tested experimentally by pycnometric means, as described below:

A sample of the mixed concrete (Table 8) of approximately 0.1-cu. ft. volume was placed in a synthetic-rubber basket-ball bladder* of approximately equal capacity when undistended. Loading was accomplished

*Darex No. B 21185 obtained from the Dewey and Almy Chemical Co.

in a manner designed to eliminate extraneous air. The bladder was corked tightly and placed on a wire-ring tripod support in a volumeter which consisted of a Pyrex vacuum desiccator (200 mm. I.D.) with a tubulated cover accommodating a No. 8 rubber stopper. The covered desiccator was filled with water until the level rose to a desired point in a burette inserted in the rubber stopper. Air trapped in the desiccator was removed by mild agitation and tapping.

The procedure of the test was arranged so that the initial reading of the water level in the burette was taken 20 minutes after the water was added to the cement in the mixture. Subsequent readings were taken by an observer at frequent intervals until the end of the working day. Readings were taken during the night and for the remainder of the 96-hr. test period at one-half hour intervals by an automatically operated moving-picture camera.

The apparatus was maintained at a constant temperature of 70 ± 2 F. throughout the test. The absence of volumetric influences caused by evolution of heat within the test specimen was observed by placing thermocouples in several test specimens and in the surrounding water. The temperature variation at the center of gravity of the specimen was inappreciable, being less than 4 F. in all cases.

Each of the curves shown in Fig. 8 represents the average of tests made in duplicate and repeated on two different days. The concordance between individual results was good up to the 40-hr. period but diverged somewhat thereafter from causes not yet determined. The data indicate marked differences in the influence of the several admixtures and the two cements on the early volume of concrete. The effect is most notable with admixture V in which the delayed generation of hydrogen gas causes an early tendency toward expansion and approximately zero net expansion at the end of the test.

Work with this test procedure will be extended both in combination of materials tested and in duration of the test. The data presented herein are of relative value only and a direct interpretation of the effects shown in terms of the volume constancy of restrained concrete cannot as yet be made.

(e) *Laboratory freezing and thawing.* The effect of the admixtures on the resistance to rapid freezing and thawing was determined with two cement contents and two cements. The mixtures were proportioned as indicated in Table 8. The 3½- by 4½- by 16-in. beam specimens were removed from the molds at an age of 48 hr. and stored in fog at 70 ± 2 F. for seven days. The beams were made in groups of three and each variable in the program was repeated on at least three different days. Freezing and thawing was commenced at an age of specimen of

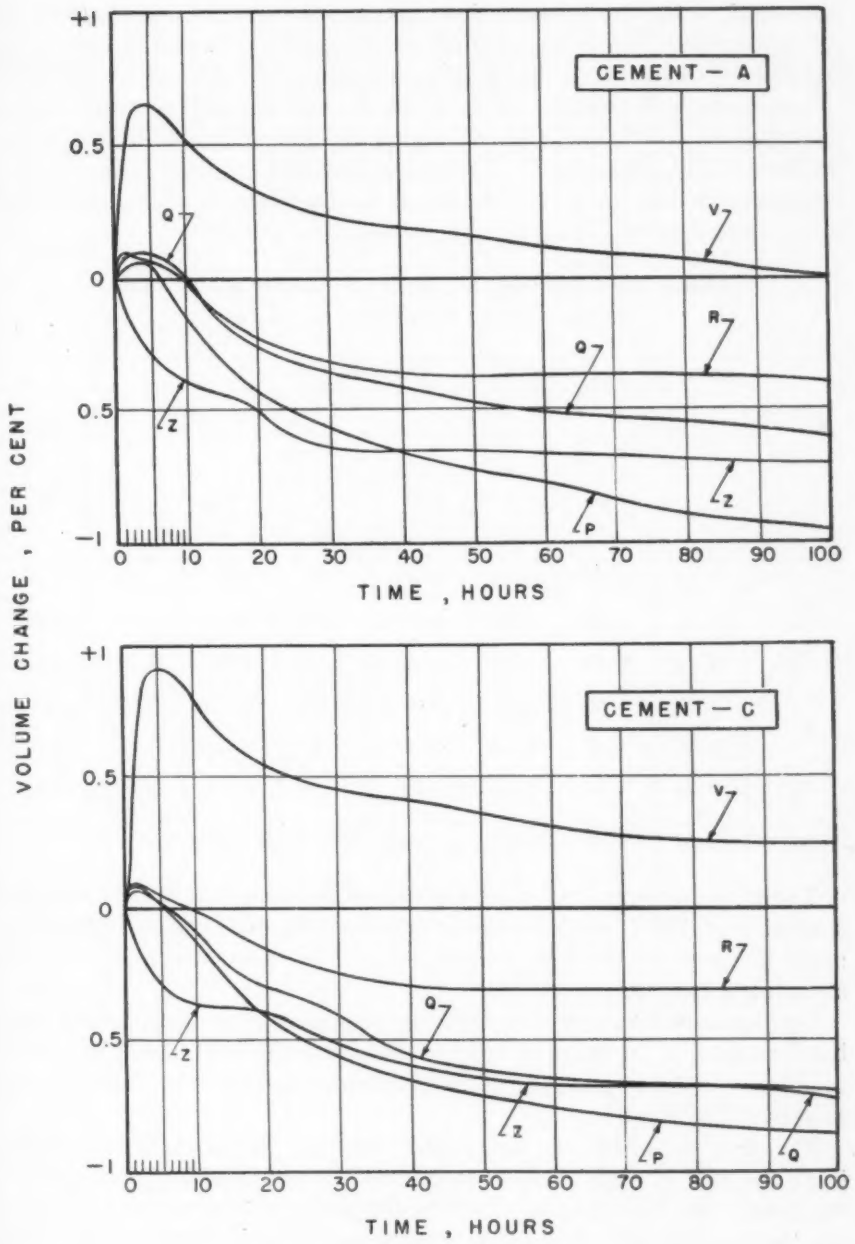

Fig. 8—Early unrestrained volume change of concrete made with and without admixtures

nine days, with the dynamic E at this age being taken as the zero or reference value (the averages of these references values of E and those for 28 days are shown in Table 20 as information). An additional series of specimens was prepared to duplicate the above in all respects except that they were subjected to freezing and thawing after a curing period of 28 days. The object of this second group was to determine whether subjecting the specimens to freezing at the early age of nine days would result in low or otherwise abnormal values for durability.

TABLE 20—DYNAMIC MODULI OF ELASTICITY—PSI.x10^6

Admix.	Nominal Cement Factor; bags per cu. yd. 4.50				6.00			
	Cement							
	A		C		A		C	
	Age at Test—Days							
	9	28	9	28	9	28	9	28
	Dynamic Moduli of Elasticity—psi x10^6							
P	4.85	5.68	4.82	5.53	5.86	6.54	6.03	6.36
Q	4.91	5.50	4.51	5.03	6.12	6.64	5.81	6.25
R	4.95	5.69	4.89	5.56	5.87	6.35	5.78	6.08
U	4.58	5.00	4.81	5.31	5.92	6.51	5.20	5.67
V	5.26	5.92	4.83	4.52	5.91	6.35	6.01	6.61
Z	5.08	5.68	5.06	5.67	5.89	6.39	6.03	6.32

Note: All values are average results for 9 specimens for each cement and admixture.

The nine-day age for commencement of freezing was chosen in order that at least 200 cycles of freezing and thawing, plus the curing period, could be accomplished in the 28-day period normally available for acceptance testing.

Concordance between the three specimens within each round and between each of the three or more rounds was sufficiently good to permit use of uncorrected values in the summarized data shown in Table 21 and plotted in Fig. 9 and 10.

The data indicate the remarkable increase in durability resulting from the use of each of the admixtures as compared to plain-cement concrete. The difference between the durability of the concrete containing the various admixtures is relatively small, with durability factors* for all admixtures except Z being above 75 at 250 cycles for the concrete cured for nine days.

*Calculated as in Fig. 1.

TABLE 21—RESISTANCE* OF CONCRETE TO FREEZING AND THAWING

(Laboratory fast cycle)

Admix.	Cement A				Cement C			
	4.5 b./cu. yd.		6.0 b./cu. yd.		4.5 b./cu. yd.		6.0 b./cu. yd.	
	Cured 9 days	Cured 28 days	Cured 9 days	Cured 28 days	Cured 9 days	Cured 28 days	Cured 9 days	Cured 28 days
P‡	3.0	2.0	6.0	8.0	5.0	4.0	28.0	20.0
Q	89.0	80.0	88.0	83.5	92.0	82.0	91.0	92.5
R	89.0	73.5	87.0	86.5	92.0	82.0	92.0	93.5
U	86.0	79.0	83.0	79.0	82.0	78.0	91.0	91.0
V	91.5	82.0	92.5	91.5	93.5	88.0	94.0	95.0
Z	63.0	57.0	76.5	52.0	69.0	49.0	82.0	62.0

*Expressed as Durability Factor at *250 cycles* based on equation D.F.E. $= \frac{PN}{M}$ (see Fig. 1).

‡P = Concrete without admixture.

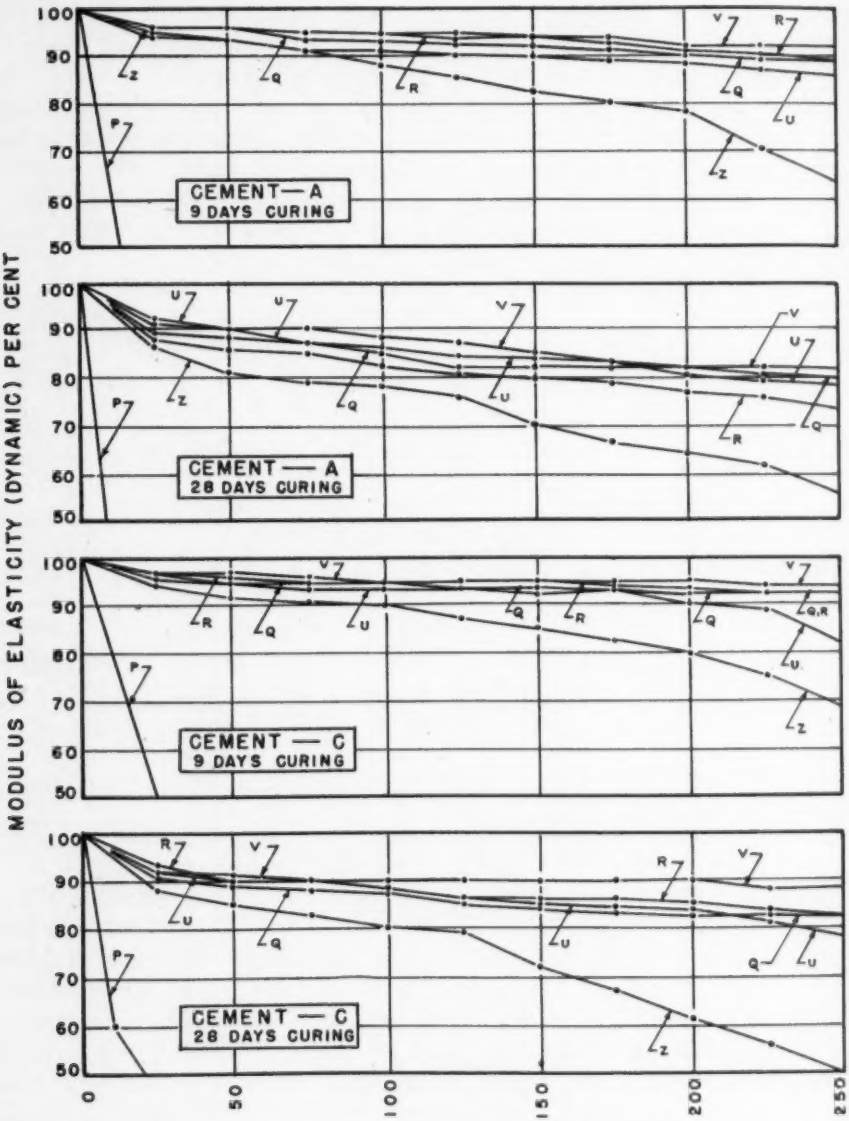

Fig. 9—Influence of admixtures on durability of concrete

Cement factor: 4.5 bags per cu. yd.

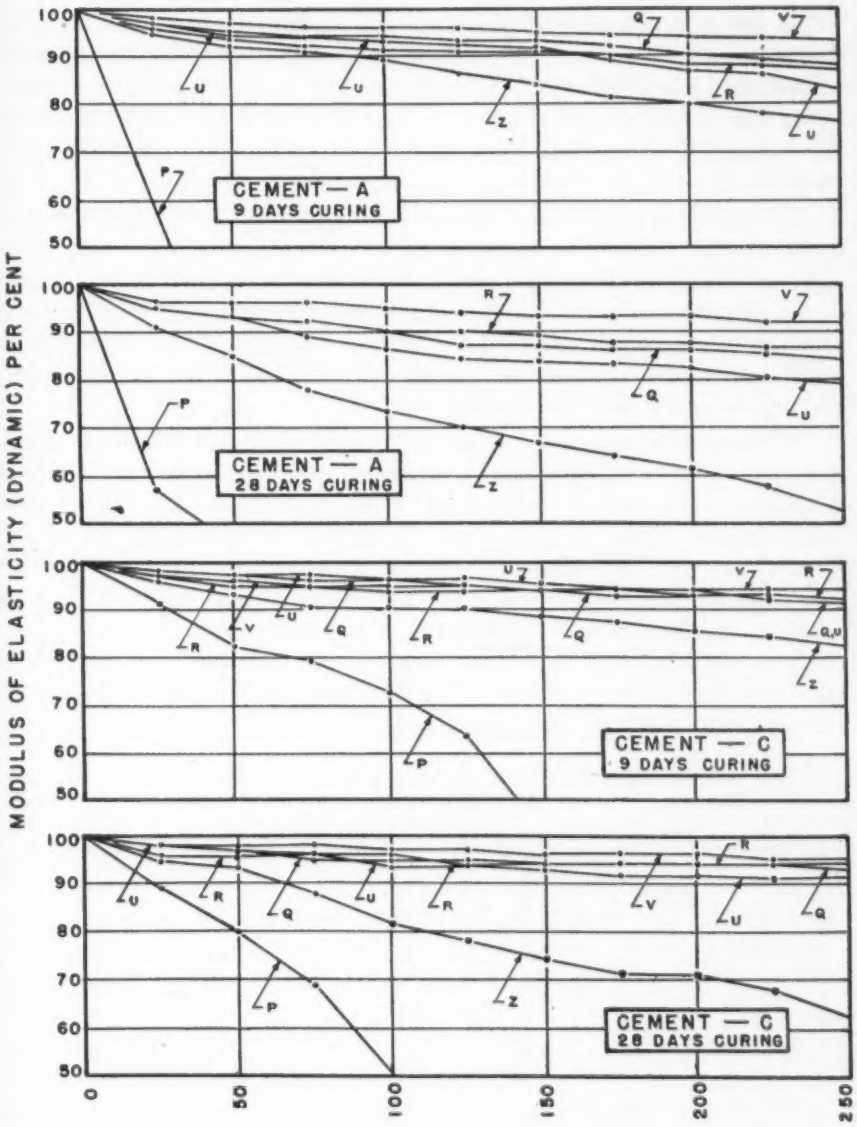

Fig. 10—Influence of admixtures on durability of concrete

Cement factor: 6.0 bags per cu. yd.

The effect of curing the specimens for 28 days instead of nine days was surprisingly small; tending toward a negative effect in all cases except the 6-bag mixtures made with cement C in which a slight positive effect was indicated with four of the admixtures. The adverse effect of the prolonged curing was most marked in all concrete containing admixture Z.

The effect of the higher cement content, with complementary reduction in W/C, was noticeable in the plain-cement mixtures with both ages of curing, but was generally inappreciable at 200 cycles in the mixtures containing the various admixtures which were cured for only nine days. The concrete cured for 28 days was moderately improved in durability at 200 cycles with reduction in W/C with the improvement being more marked with cement C than with cement A.

The large number of specimens to be tested prevented the number of cycles of the test being extended beyond 260 cycles except in a few cases toward the end of the program when it was found possible to continue certain of the specimens to a decrement in dynamic E of 50 per cent. These few data shown in Fig. 11 represent average values for the fourth and fifth rounds made with admixture V and cement A. The effect of the increased W/C, concomitant with reduced cement factor, becomes marked with prolonged freezing and thawing; the durability factor for the 4.5-bag concrete being 26 as compared to 49 for the 6-bag concrete, with both factors based on 1000 cycles.

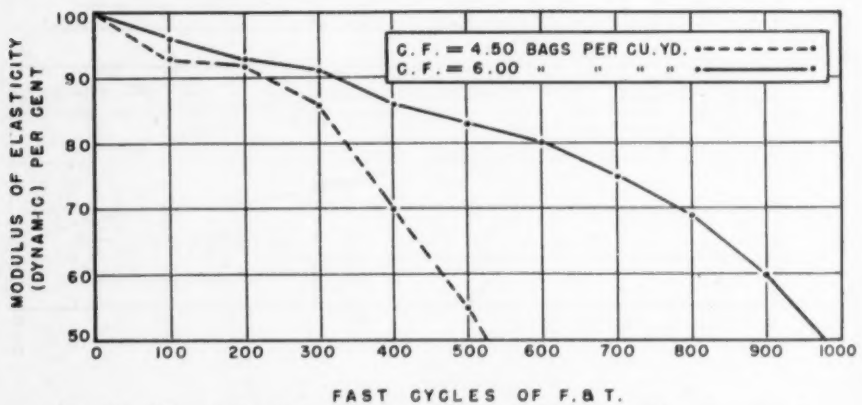

Fig. 11—Ultimate durability of concrete containing admixture V and cement A

A supplementary series of tests were made with admixture V and cement F* for the express purpose of determining the effect of W/C on durability of otherwise similar concrete. The results to 400 cycles are

*A cement meeting A.S.T.M. Specification C 150, Type I, not used previously in this work.

shown in Fig. 12 in which each curve represents the average of nine specimens made in groups of three on three different days. The anticipated influence of W/C on durability is apparent, but the full degree of difference was not developed at 400 cycles when the data were compiled for this paper.

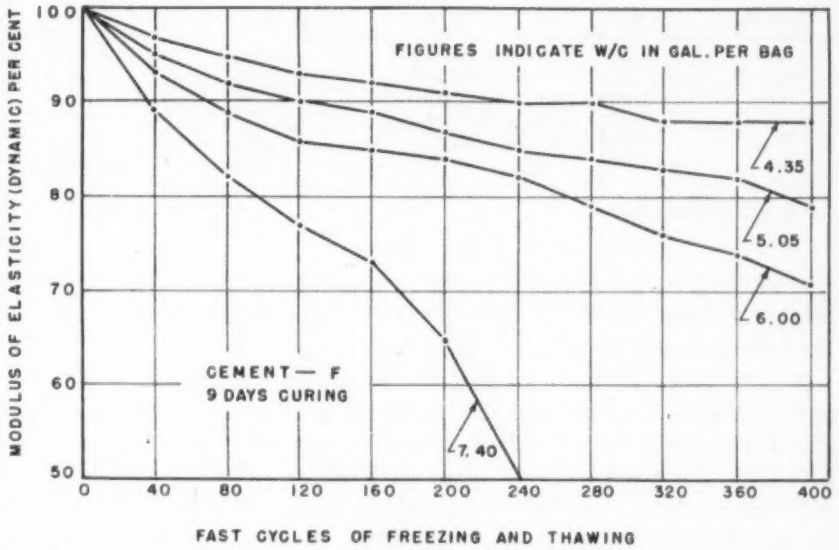

Fig. 12—Influence of variation in water—cement ratio on durability of concrete containing admixture V

(f) *Thermal properties.* The influence of the admixtures on the thermal coefficient of diffusion of concrete proportioned as shown in Table 8, appears to be entirely negligible as indicated by the results given in Table 22. It is evident that the presence of minute well distributed air voids should reduce the rate of heat diffusion, but this effect is not marked when the amount of air entrained is of the order which is optimum for the best results in concrete.

Discussion of results—supplemental studies

(a) *Effect of superimposed load.* The effect of a load equal to 2 psi imposed on the plastic concrete immediately after molding was studied by making duplicate cylinders, beams, and prisms from the batches of concrete (Table 8) made for determining compressive and flexural strength, bond-to-steel, and durability. Platens were made to fit each of these types of specimens and metal cans were prepared containing steel shot in quantities sufficient to apply a uniform load of 2 psi to the exposed surface of each specimen. Immediately after molding, the load

TABLE 22—THERMAL DIFFUSIVITY OF CONCRETE CONTAINING AIR-ENTRAINING ADMIXTURES

Diffusivity in sq. ft. per hr.

	Cement A		Cement C	
	Cement Factor—bags per cu. yd.		Cement Factor—bags per cu. yd.	
Admix.	4.5	6.0	4.5	6.0
P*	0.035	0.034	0.035	0.035
Q	0.034	0.033	0.033	0.032
R	0.034	0.032	0.035	0.032
U	0.033	—	0.033	—
V	0.034	0.032	0.035	0.032
Z	0.037	0.034	0.035	0.034

*Concrete without admixture.

was applied with a minimum of impact or vibration and permitted to remain in place for 24 hr. The effect of the loading is similar to testing the bottom third of specimens 3 ft. deep.

The influence of such loading on the strength, bond, and durability of the concrete made with and without the admixtures is shown in Table 23. It is evident from the data that the properties tested were all benefitted by the superimposed load, with the benefit varying markedly with the admixture used and with plain cement. The property benefitted to the greatest degree was bond-to-steel. Concrete containing admixture V, having developed an extraordinary improvement in the normal test (see Table 17) showed a minimum improvement in bond-to-steel under the loaded condition. Durability was the property least benefitted by the loaded condition.

(b) *Effect of vibration on air content.* The air content of concrete mixtures is determined normally by comparing the weight per cu. ft. of the plastic mixture with the theoretical unit weight in accordance with A.S.T.M. Method C 138-44. In this method, the plastic unit weight is determined by hand-rodding the mixture in the container. In view of the prevalence of consolidation of concrete by internal vibration in the field, it is important to know the effect of vibration on the air content as determined by the standard method. This problem was investigated by making a series of 20 mixtures with plain cements and with admixture V, and determining the unit weight in 1 cu. ft. measures simultaneously by the standard rodding method and by internal vibration. The air contents were determined by calculation and the average values obtained are given in Table 24. The vibration was continued beyond the time which would be optimum in the field for the mixtures used in order to accentuate rather than minimize the effect thereof on air content.

TABLE 23—EFFECT OF LOADING ON COMPRESSIVE, FLEXURAL, AND BOND STRENGTH, AND ON DURABILITY OF CONCRETE

(Load = 2 psi)

Admix.	Expressed as Per Cent of Value Obtained for Comparable Unloaded Concrete (See Tables 13, 16, 17, and 21 for unloaded values)									
	Cement A					Cement C				
	Comp. Str. 2 days	Comp. Str. 28 days	Flex. Str.	Bond-to-Steel	Durability D.F.E.	Comp. Str. 2 days	Comp. Str. 28 days	Flex. Str.	Bond-to-Steel	Durability D.F.E.
				Cement Factor = 4.5 b./cu. yd.						
P*	119	109	112	136	105	110	102	108	117	109
Q	112	102	104	208	102	106	104	108	172	103
R	113	103	112	216	103	120	107	116	163	103
U	115	105	105	203	101	116	103	107	168	101
V	108	99	117	153	110	118	107	114	174	112
Z	110	105	100	127	101	118	103	103	196	100
				Cement Factor = 6.0 b./cu. yd.						
P*	100	107	109	165	103	116	106	103	170	106
Q	110	110	103	176	102	109	109	111	182	102
R	109	106	101	234	101	106	108	98	147	102
U	103	113	107	140	100	110	101	111	140	100
V	116	117	106	117	108	125	116	110	120	108
Z	107	101	109	129	102	103	106	108	127	99

*P = Concrete without admixture.

TABLE 24—EFFECT OF VIBRATION ON AIR CONTENT OF CEMENT

Air Content in Per Cent

Plain Cement			Admixture V		
Rodded	Vibrated	Difference	Rodded	Vibrated	Difference
1.3	1.0	−0.3	3.3	2.9	−0.4
2.1	1.5	−0.6	3.7	3.2	−0.5
2.5	1.8	−0.7	3.9	3.6	−0.3
2.8	1.9	−0.9	4.2	3.7	−0.5
3.4	2.4	−1.0	4.4	3.7	−0.7

The data indicate that vibration does result in an apparent reduction in air content, with the degree of reduction being greatest in plastic mixtures of plain-cement concrete having relatively high air contents. This phenomenon is believed to be explained in part by the nature of the voids formed in plain concrete and in concrete with purposefully entrained air. The air entrained in plain concrete is present usually in relatively large voids of sufficient volume to develop buoyancy in sufficient degree to cause rapid upward migration under the influence of vibration. However, the widely distributed minute voids formed under the influence of a sudsing agent have inadequate volume to exert effective buoyant force. Little evidence of upward migration of the minute voids have been noted in hardened concrete under the microscope, whereas, the migration of large air voids under the influence of vibration is very evident to the naked eye in plain concrete. Therefore, it may be possible that the apparent reduction in air content in the concrete-with-admixture may be caused largely by loss of the large bubbles of incidentally entrapped air typical of plain concrete which are present also, in purposefully air-entraining concrete. A second partial explanation for apparent reduction in air content may be due to a compression of the air under the fluid pressure developed under effective vibration. It has been observed, too, that the unit weight of hardened concrete, as measured by displacement in water, is usually greater than that of the same plastic concrete. This reduction in apparent air content may be due in part to shrinkage of the concrete, evaporation of moisture, and by partial filling of some of the air voids with water.

In any case, the apparent reduction in air content caused by vibration is not sufficient in degree to cause alarm over the possible reduced effectiveness of air entrainment on the desirable properties of concrete. This has been demonstrated definitely by the fact that all concrete specimens reported in this paper were consolidated by thorough vibration and the beneficial effects of the purposeful air entrainment are very apparent in the freezing and thawing test.

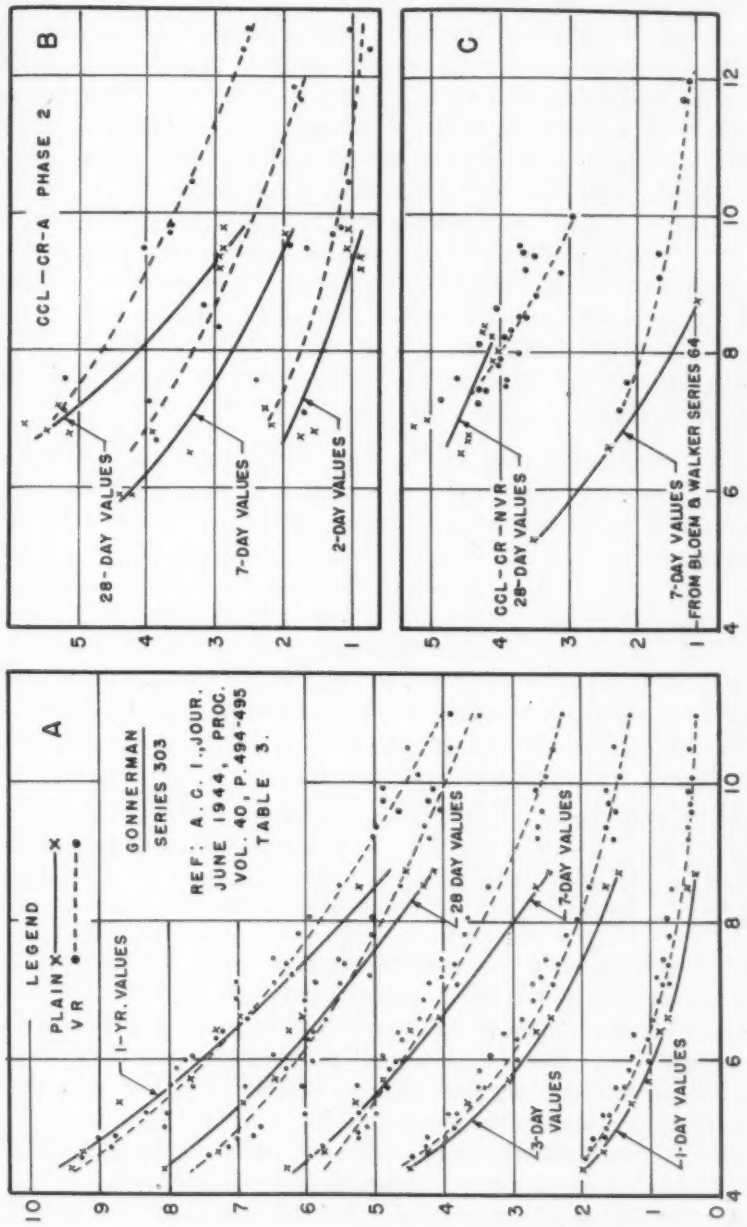

Fig. 13—Relation of total volume of water plus air in concrete to compressive strength at various ages of test

(c) *Effect of total volume of air plus water on compressive strength.* It has been suggested (47, 83) that the effect of entrained air in concrete on compressive strength is to cause a reduction similar to that which would have been induced by an increase in the mixing water in a volume equal to the volume of the entrained air. Gonnerman (47) shows a curve for voids-cement ratio vs. compressive strength at 28 days in which the curve represents both plain and air-entraining cement concretes and for which the factor "voids" represents absolute volume of air plus water. Since this curve was constructed on the basis of a plot using only 20 of the values given in his Table 3 at but one age, a new graph was drawn using 42 values for each of five test ages (Fig. 13A). Similar graphs were constructed for data developed in work conducted at this laboratory with admixture R (Fig. 13B and C). Also shown on Fig. 13C are the curves given in Fig. 3 of a paper by Bloem and Walker (83).

The relationships suggested by the data plotted in Figs. 13A and B have been indicated by curves. In both cases an increased air plus water volume resulted in an appreciably greater reduction in strength with plain cement than when an air-entraining admixture was used, except for low values of W + A/C at ages of seven days and over. A similar effect is suggested by the curves from Bloem and Walker. The curves in Fig. 13C for work done by this laboratory fail to show this relationship, due, principally, it is believed, to the fact that a number of different brands of cement were used with different coarse aggregates, under circumstances in which there was very little variation in W/C for any one combination.

In general, it is believed that there is an indication from these data that, except for low values for W + A/C at ages greater than seven days, air has less effect than water, in equal volume, on compressive strength of concrete.

(d) *Effect of entrained air on reduction in unit weight of concrete.* One of the methods which has been applied in certain specifications as a limiting requirement for the air to be entrained in concrete has been that the unit weight of the concrete shall be reduced by a specified number of pounds per cubic foot from that which would be obtained with plain concrete. This method leaves something to be desired because it does not take into account the effect of possible differences in the bulk specific gravity of the fine and coarse aggregate and the reduction in sand-aggregate ratio and W/C made practicable and desirable by purposeful entrainment of air. The data in Table 25 demonstrate the effect which different combinations of aggregates have on reductions in unit weight with air contents varying from 0 to 5 per cent.

TABLE 25—VARIATION IN UNIT WEIGHT OF CONCRETE AS AFFECTED BY AIR CONTENT AND SPECIFIC GRAVITY OF AGGREGATES

Bulk Sp. Gr.		W/C g.p.b.	Cement Factor bags per cu. yd.		S/A % by Vol.	Actual Unit Wt. lb. per cu. ft.	Air %	Reduc. in Unit Wt. lb. per cu. ft.
Sand	C. Agg.		Theo.	Actual				
2.50	2.90	6.00	4.50	4.50	35.0	159.4	0.0	0.0
2.50	2.90	5.80	4.54	4.50	33.7	158.6	1.0	0.8
2.50	2.90	5.50	4.59	4.50	32.4	157.9	2.0	1.5
2.50	2.90	5.25	4.64	4.50	31.1	156.9	3.0	2.5
2.50	2.90	5.00	4.68	4.50	29.8	156.1	4.0	3.3
2.50	2.90	4.75	4.72	4.50	28.5	155.2	5.0	4.2
2.65	2.65	6.00	4.50	4.50	35.0	154.2	0.0	0.0
2.65	2.65	5.80	4.54	4.50	33.7	153.0	1.0	1.2
2.65	2.65	5.50	4.59	4.50	32.4	151.9	2.0	2.3
2.65	2.65	5.25	4.64	4.50	31.1	150.9	3.0	3.3
2.65	2.65	5.00	4.68	4.50	29.8	149.9	4.0	4.3
2.65	2.65	4.75	4.72	4.50	28.5	148.8	5.0	5.4
2.65	2.50	6.00	4.50	4.50	35.0	149.3	0.0	0.0
2.65	2.50	5.80	4.54	4.50	33.7	148.1	1.0	1.2
2.65	2.50	5.50	4.59	4.50	32.4	147.0	2.0	2.3
2.65	2.50	5.25	4.64	4.50	31.1	145.8	3.0	3.5
2.65	2.50	5.00	4.68	4.50	29.8	144.8	4.0	4.5
2.65	2.50	4.75	4.72	4.50	28.5	143.5	5.0	6.0

FUTURE WORK

Mass concrete

The major use of air entrainment in concrete to date has been in pavements and relatively non-massive structures. However, the major contribution of purposeful air entrainment to increased plasticity, water-retentivity, and durability should not be ignored in connection with massive structures. It is apparent from all work performed to date that the leaner mixtures are benefitted more than are the richer mixtures by the use of air entrainment. It is apparent, too, that the very lean mixtures containing large aggregate used in massive concrete structures are relatively non-plastic and possess a low order of coherence. It is at least reasonable to expect that the plasticizing effect of air entrainment should improve the mobility and coherence of such mixtures. Pilot studies conducted in this laboratory in recent months, in which ½-cu. yd. blocks of lean concrete containing 4-in. and 6-in. aggregate have been cast, cored, and tested, indicate major improvements in placeability, coherence, and durability with no sacrifice in strength and *minor reductions in unit weight.* In cases where the admixture is added at the mixer, very close regulation and control of the amount of entrained air is thoroughly practicable.

(a) *Amount of air desired.* One of the principal features of the application of air entrainment to mass concrete mixtures is the fact

that the total amount of entrained air need be less than in pavement or structural concrete mixtures for equal effect, because use of considerably larger coarse aggregate results in a reduction in the mortar component of the mass. It is the condition and air content of the mortar which affects the mobility, coherence, and durability of the concrete. It can be demonstrated that a total air content of 2.7 per cent in lean concrete mixtures containing 6-in. aggregate is comparable insofar as the mortar constituent is concerned to a total air content of 4.0 per cent in a pavement mixture containing 1½-in. aggregate. For this reason it is practicable to achieve full benefits from air entrainment in mass concrete with a minimum of reduction in unit weight.

(b) *Determination of unit weight of mass concrete.* The preliminary studies referred to above have included tests of unit weight. In these studies, *it was demonstrated that the usual procedure of wet-screening to remove aggregate larger than 1½-in. to permit use of standard containers was impractical and produced incorrect results.* The process of passing a concrete mixture over a sieve separates the ingredients and exposes a maximum of mortar surface to the air. The agitation of the separate mixture permits the loss of a considerable portion of the entrained air so that the anticipated relationship of air to the residual concrete mixture smaller in size than the sieve used is not obtained. Therefore, it is considered necessary to determine the air content of all mixtures with a minimum of disturbance and without wet-screening. When aggregate larger than 2 in. is used, it becomes necessary to determine unit weight in 2-cu. ft. measures. Such tests are being conducted currently in the laboratory.

It is proposed to expand the studies of purposeful air entrainment in lean concrete mixtures containing large aggregate sizes with the object of developing means for specifying and controlling air entrainment in massive structures.

GENERAL DISCUSSION

The data presented in this paper are summarized below with an interpretation of their significance and application to the successful use of air entrainment in concrete.

Action of air-entraining admixtures

Air-entraining admixtures will cause the formation of a foam composed of small to minute bubbles of air when shaken vigorously with water by reduction in the surface tension of the solution. The quantity, dispersion, and stability of the foam is increased in a mixture of sand, water, and a given amount of admixture as the quantity and coarseness (within the usual limits of fine aggregate) of the sand and the time and

vigor of mixing are increased. The addition of cement to the mixture, with proportional reduction in sand, results in a decrease in amount of foam as the ratio of cement to sand increases to the ultimate point where very little air is entrained in a simple cement-water-admixture system. Therefore, cement may be considered to be a depressant of air entrainment. In a concrete mixture, the amount of air entrained varies inversely with the cement content and directly with the sand-total aggregate ratio. Further, the amount of air entrained increases with increased slump or flow of the mixture. The entrainment of air in concrete may be accomplished most satisfactorily when the admixture is not dependent upon chemical reaction with the cement to develop the requisite sudsing property.

From the foregoing, it is apparent that the term "air-entraining cement" is a misnomer, although evolved naturally from the fact that air-entraining admixtures were introduced to modern concrete practice as interground additions to cement.

Regulation of the amount of air entrained

The close regulation of amount of air to be entrained in a mortar or concrete mixture can be accomplished most practically by adjustment of the quantity of the given type of admixture used; as regulation by varying the cement or sand content, the time and type of mixing, or the consistency of the mixture is limited by economics, structural design factors; and available mixing, transporting, and placing equipment. When the admixture is interground with the cement, factors of adsorption, carbonization, volatilization, and metamorphism caused by the heat of grinding affect the quantity of the admixture necessary to be added to develop a given amount of air entrainment in a mortar or concrete mixture. By use of a standardized mortar test (A.S.T.M. C 185-44T), it is possible to regulate the amount of admixture necessary to entrain a given amount of air in that mortar and, when the admixture is interground with the cement, this method of control is superior to quantitative limitations based on chemical analysis. However, varying factors of cement content; amount and grading of sand; shape, grading, and size of coarse aggregate; time and type of mixing, and consistency vitiate largely the value of the standard mortar method as a means of predicting or controlling the air content in concrete mixtures.

Effect of air-entraining admixtures on the properties of concrete

The effect of the use of the so-called "air-entraining admixtures" on the properties of concrete appears to be a function of the amount and condition of the air entrained, that is; the number, size, and degree of distribution of the bubbles of air in the mortar component of the mixture, rather than on total volume alone. The chemical effect of these

admixtures appears to be limited entirely to the possible presence and type of other than sudsing compounds, such as accelerators, deflocculators, or gas-generating agents. Considering only the sudsing agent, the effect of entrained air is cumulative, as indicated below:

(a) The entrained air acts as a very elastic and stable non-reactive fine aggregate of high lubricating value. Its presence permits:

(b) A marked reduction in the water-cement ratio necessary to produce the desired placeability of the mixture, and

(c) A reduction in the sand-total aggregate ratio normally required by approximately 1.3 times the amount of air entrained; thereby reducing the total surface area of rigid aggregate to be coated and lubricated by the cement-water paste.

(d) Reduction in the W/C effects a basic increase in the strength and durability of the cementing medium, and the reduction in the total water present in the mixture reduces the amount of excess water available for formation of channels through the matrix of the concrete; thereby reducing permeability and bleeding.

(e) Reduction in bleeding is increased beyond that effected simply by reduced W/C by immobilization of additional water through adsorption on the air bubbles. Reduction in bleeding results in diminished separation of the matrix from the under sides of coarse aggregate particles and in diminished flotation upward of laitance.

(f) Finally, the numerous well dispersed air voids provide reservoirs for the relief of pressure created in concrete due to differential volume movements in the concrete caused by temperature change and by the expansion accompanying the transition of water to ice. This contribution to durability is reinforced by the reduced W/C and lack of channelization of the matrix due to bleeding.

It appears that the entrained air is more closely related to the fine aggregate than to any other component of a concrete mixture and that the benefits inherent in purposeful air entrainment are related principally to that constituent of the mixture. Therefore, the optimum percentage of air entrained should be a function of the quantity of fine aggregate present in the mixture rather than a fixed percentage of the total mixture. In rich mixtures containing coarse aggregate of small maximum size, the total amount of air to be entrained for optimum results should be considerably greater than would be required in lean mixtures containing large coarse aggregate, because the sand component of the former is much greater in amount than in the latter.

(a) *Effect on strength and W/C.* From data which is based preponderantly on tests made with Vinsol resin interground with cement

at the mill and concrete containing 1½-in. coarse aggregate (65, 71, 79, 82) the air entrained in concrete results in a reduction in strength which is related roughly in amount to that which would accompany an equal increase in volume of water, that is, the volumetric ratio of water plus air to cement is roughly but not quite equal to the volumetric ratio of water to cement in its effect on strength. Since the ratio of practicable reduction in W/C to increase in air content is reduced sharply beyond approximately 4 per cent air, increases in air content beyond that amount are accompanied by a rapidly increasing reduction in strength. Since a maximum increase in durability appears to be reached when the air content is approximately 4 per cent and the reduction in strength is not appreciable at that point, this amount of air entrainment is considered to be the optimum for use in concrete mixtures containing coarse aggregate up to about 1½ in. in maximum size.

The sacrifice in compressive and flexural strength and in bond of concrete to steel which is associated usually with the use of simple air-entraining admixtures in concrete may be minimized by addition of the admixture at the mixer where the quantity added may be regulated at will and controlled closely to produce the optimum air content. Data from the tests made with admixtures which are compounds of sudsing agents and various accelerators, deflocculators, and gas-generating agents indicate that the full benefits of the entrainment of air may be retained with an accompanying definite increase in strength and bond-to-steel. As would be expected, the degree to which the compound admixture affects the properties of the plastic and hardened mixture is a function of the composition of the admixture.

The benefits to concrete quality which appear to be available, potentially, through the use of compound air-entraining admixtures appear to be very great, but the competitive nature of such admixtures and the possible adverse influence of minor quantities of certain materials on concrete quality make evident the necessity for adequate laboratory performance tests to govern their acceptability for use. In recognition of these benefits and possible liabilities, a specification for air-entraining admixtures and a method of evaluating their performance in concrete have been formulated and submitted for the approval of the Department. They are not yet available for publication.

(b) *Effect on other properties.* When added to the concrete at the mixer so as to produce the optimum quantity of entrained air, the use of air-entraining admixtures will not result in *increased* retardation of set or early strength gain at temperatures above freezing. It is possible to achieve increased early strength at low temperatures by using a satisfactory accelerator with or as a part of the admixture with no sacrifice in the desirable properties of concrete.

Purposefully entrained and properly dispersed air will not be displaced upward or escape to a material degree from well designed concrete mixtures consolidated by proper application of vibration.

The thermal properties of concrete are not affected to a material extent by the entrainment of optimum quantities of air.

The volume constancy of concrete, due to moisture or temperature changes, is not affected materially by the entrainment of optimum quantities of air.

The degree to which the air content of a mixture will increase is a function of the nature of the admixture and the immediacy of its availability as a sudsing agent when introduced in the mixer. The entrainment of air is not likely to increase in concrete mixtures by continued mixing in stationary or paving-type mixers beyond a period of about three minutes when the admixture is added at the mixer. Continued increase in air with prolonged mixing is likely to occur beyond this period when the admixture is interground with the cement at the mill.

The air content of concrete resulting from the use of air-entraining admixtures should be considered as a definite ingredient and the design of mixtures should be based on ingredients of water, *air*, cement, fine and coarse aggregate.

In view of the benefits to be derived from mixtures of air-entraining agents, accelerators, and gas-generating agents, and the regulability of effect achievable by such use, the admixture should be added to the concrete mixture in the field at the batching plant or mixer.

The addition of admixtures should be made by mechanical batchers which will dispense the material in accurate and regulable amounts.

ACKNOWLEDGMENT

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Shrinkage Stresses in Concrete*

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Member American Concrete Institute

PART 2—APPLICATION OF THE THEORY PRESENTED IN PART 1 TO EXPERIMENTAL RESULTS

Carlson's results on prisms drying from one end

As mentioned in Part 1, Carlson[1] applied diffusion principles to the problem of computing both loss of moisture and distribution of shrinkage. The fundamental equations on which his computations were based are the equations to which Equations 5 and 22 of Part 1 reduce when the parameter B is set equal to infinity. In his experimental work the prisms were allowed to dry through one end only, the rest of the surface being sealed. Measurements were made over gage lines that were parallel to the direction of flow of moisture. These conditions appear to be most favorable for the direct measurement of the distribution of shrinkage tendency since in an unrestrained specimen shrinkage stresses should not have any appreciable effect on the unit shortening in the direction of moisture flow.

In Fig. 3 of his paper Carlson showed two diagrams. One diagram gave the distribution of shrinkage as measured after a definite period of drying and the other gave the computed "distribution of drying" (loss of moisture) for different assumed coefficients of diffusion for the same period of drying. The observed distribution of shrinkage and the computed "distribution of drying" are in good agreement when the proper coefficient is selected. However, as shown by Fig. 1 of his paper, the measured loss in weight was not in very good agreement with the theory. Carlson could have obtained slightly better agreement between theory and measured shrinkage if he had taken surface conditions into account,

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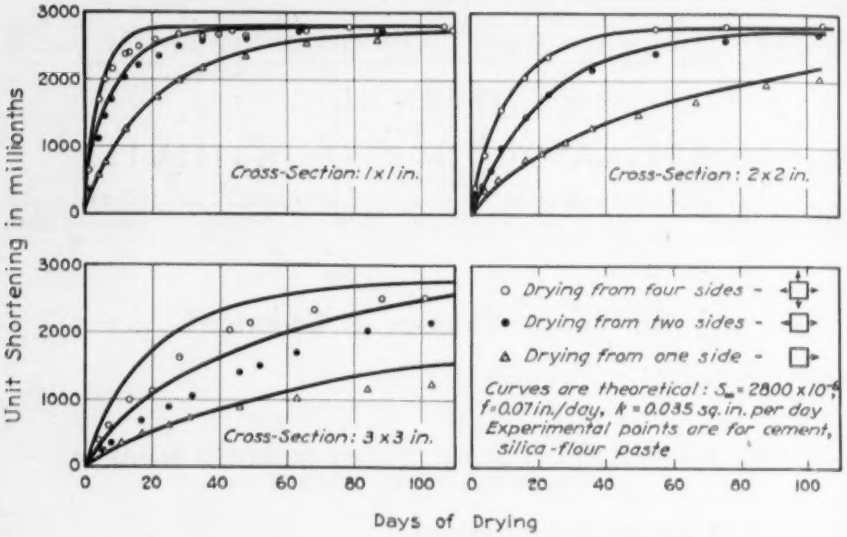

Unit Shortening in millionths
3000
2000
1000
0
Cross-Section: 1x1 in.
Cross-Section: 2x2 in.
Cross-Section: 3x3 in.
o Drying from four sides -
• Drying from two sides -
Δ Drying from one side -
Curves are theoretical: S_∞ = 2800 x 10⁻⁶, f=0.07 in./day, k = 0.035 sq. in. per day
Experimental points are for cement, silica-flour paste
0 20 40 60 80 100
Days of Drying

Fig. 15—Comparison of observed and calculated course of shrinkage

i.e., used a finite value for the parameter B. However, had he done so, the discrepancy between theory and measured loss in weight would have been greater than that shown.

Carlson's work is important evidence in support of the hypothesis that shrinkage of concrete approximately follows the laws of diffusion.

Shrinkage of prisms of various sizes drying from one or more sides

In the work done in this laboratory measurements were made on gage lengths transverse to the direction of moisture flow. Since variations in shrinkage along the path of moisture flow result in stresses transverse to the direction of flow, the measurements include the strains produced by these stresses. If, however, the specimens are long compared to their dimension in the direction of moisture flow and the stress-strain relation is linear, then as shown in Part I the shortening of the central axis will be equal to the average shrinkage. The results to be discussed provide a test of the theory for conditions in which both size of specimen and number of exposed sides are variable.

Fig. 15 shows the unit shortening versus days of drying for three different sizes of prisms made of the same mix and for the three different drying conditions discussed in Part 1. Mix A and cement M, described in the Appendix, were used. The specimens were cured seven days under water. Each point is the average of the results from two prisms. The curves were constructed from computations based on the theoretical equations developed in Part 1. These equations, which give the theoreti-

cal relationship between unit shortening, the constants of the material, and dimensions of the specimen have the form

$$\text{unit shortening} = S_{av} = F\left(S_\infty, \frac{fb}{k}, \frac{kt}{b^2}, \frac{c}{b}\right)$$

where S_∞ is ultimate shrinkage for the assumed final drying, f is the surface factor, k is the diffusivity factor, and b and c are dimensions of the specimen. The exact form of the function, especially the way in which c/b enters into it, differs with the drying condition.

The three constants S_∞, f, and k were evaluated from average experimental values for the pair of prisms of 2-in. square cross-section, drying from four exposed sides. From these same constants the curves were constructed, as shown in Fig. 15, not only for this pair but also for the theoretical unit shortening of the other eight pairs of prisms. The agreement between the experimental values and the calculated curves is fairly satisfactory except for two pairs of 3x3-in. specimens, which were observed to have cracked during drying and therefore could not be expected to shorten in accordance with the theory.

Discussion of the validity of the theory on the basis of the foregoing data

The data from those specimens that did not crack, together with the data given by Carlson, might seem to indicate rather conclusively that shrinkage does take place in accordance with the theory developed in Part 1. However, such a conclusion would not be justified. A good fit between an equation and experimental data is necessary but it is not sufficient proof of a theory. Although constants in the equations of Part 1 may be chosen so that the theory given there will be in good agreement with experiment for certain measurements on specimens under a few different conditions, the theory should be expected to fail under some other conditions since it rests on some assumptions that are not wholly correct. Shrinkage is not linearly related to change in moisture content; the flow of moisture in concrete does not follow the law of diffusion; and the stress-strain relation is not linear. Since the assumptions are not wholly correct, the factors S_∞, f, k, and E that are supposed to characterize the material must be empirical, and experimentally determined numerical values of these factors will be different for different tests on the same material. The good agreement between theory and the experimentally determined contraction of the specimens discussed above must be the result of the balancing of opposing effects. They will not necessarily balance the same way in another test.

The foregoing criticism means that however promising the theory may appear from the results of a few experiments, the application of the theory must be limited and extrapolation of the results to sizes of specimens or conditions of drying other than those for which the constants

were determined cannot be made with confidence. The selection of these constants for given conditions constitutes the chief difficulty for the practical use of the equations. This does not mean that the theory is of little value; we believe it to be of considerable value.

True, the theoretical equations are not rigorously correct and the constants cannot have exactly the meaning attached to them. But the evidence is that shrinkage does follow the diffusion equation approximately and that the deformations and stresses are approximately those given by the theoretical equations if the empirical constants selected are such as to give fair agreement with experimental results.

Although the experimental results shown in Fig. 15 appear to be in good agreement with the theory, a close study shows the following in regard to those prisms drying from only one side: (1) After one or two months of drying the shortening of the prisms drying from one side becomes progressively less than that indicated by the theoretical curves. (2) The experimental results from the 2x2-in. prisms deviate more from the computed values than do those of 1x1-in. cross-section. (The 1x1-in. specimens drying from one side have the same b-values as the 2x2-in. specimens from which the constants S_∞, f, and k were determined.) (3) The 3x3-in. prisms drying from one side deviate still more from the computed values than do those of 2x2-in. cross-section. It is expected that extrapolation to still larger sizes would result in still greater discrepancies between experimental and computed values unless allowance is made for change in the constants with change in b-value.

Comparison of data on warping with data on shortening

A more critical test of the theory is provided by the results shown in Fig. 16. The abscissa for these diagrams is the square root of the period of drying divided by the thickness of the prism.* As mentioned in Part 1, using square-root-of-time as the abscissa gives a nearly straight line for an appreciable portion of plots of both shortening and warping. Dividing the square root of time by the thickness of the specimen puts all specimens on a more nearly comparable basis. Multiplying the warping (deflection of center of a 32-in. span) by the thickness puts the "free warping" specimens on the same basis in regard to unit deformation.

The plotted points in the upper diagram of Fig. 16 represent the shortening of those prisms of Fig. 15 that were drying from only one side. The points in the lower diagram represent the warping of the specimens made of the same kind of concrete, also drying from only one side. Although these two sets of data were not obtained on the same specimens at the same time, they are representative of what is obtained when both sets of measurements are made simultaneously on the same specimens.

*The dimension in the direction of drying is taken as the thickness.

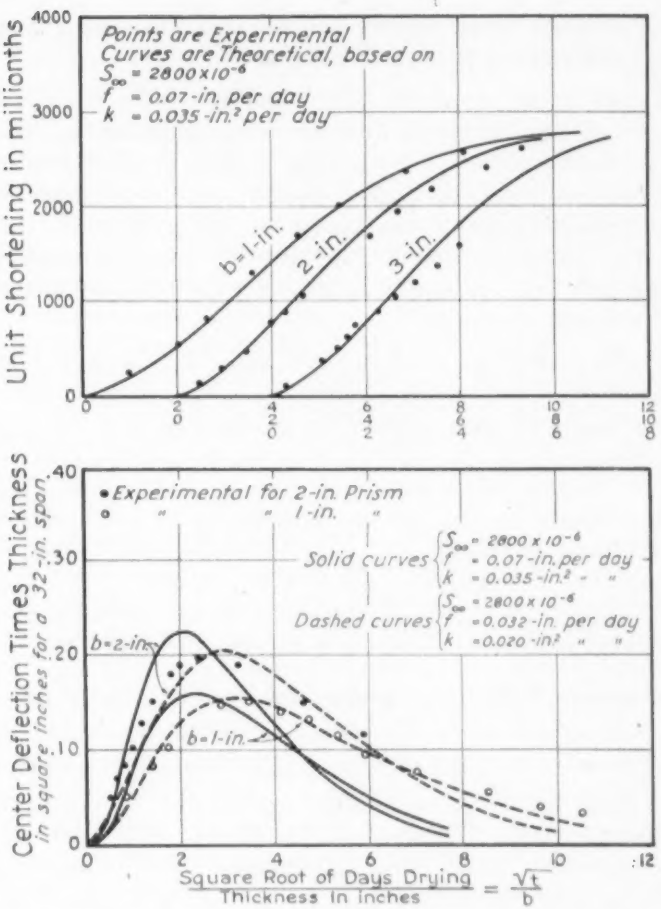

Points are Experimental
Curves are Theoretical, based on
S_∞ = 2800 x 10⁻⁶
f = 0.07 -in. per day
k = 0.035 -in.² per day
Unit Shortening in millionths
b=1-in.
2-in.
3-in.
Center Deflection Times Thickness
in square inches for a 32-in. span
•Experimental for 2-in. Prism
o " " 1-in. "
Solid curves S_∞ = 2800 x 10⁻⁶, f = 0.07 -in. per day, k = 0.035 -in.² " "
Dashed curves S_∞ = 2800 x 10⁻⁶, f = 0.032 -in. per day, k = 0.020 -in.² " "
b=2-in.
b=1-in.
Square Root of Days Drying / Thickness in Inches = √t / b

Fig. 16—Comparison of observed and calculated course in shortening and warping of prisms of mix A and cement M

The curves in the upper diagram and the solid curves in the lower diagram were constructed from the theoretical equations using the same values of the constants S_∞, f, and k as were used in constructing the curves of Fig. 15. Therefore, the three curves in the upper diagram of Fig. 16 represent the same equations as three of the curves in Fig. 15; they differ only in the abscissas. The solid curves in the lower diagram deviate considerably from the plotted points. This indicates even greater disagreement with theory than is shown in Fig. 15. However, the dashed curves obtained from the theoretical equation by using the same value of S_∞ but with f reduced by 54 per cent and k reduced by 43 per cent are in very good agreement with the experimental values.

The computed shrinkage stresses will be about the same whether the first or the reduced values of f and k are used.

The values of the constants S_∞, f, and k, used in constructing the dashed curves, were obtained from three measurements as follows: (1) maximum warping of the 1-in. specimen, (2) time at which this maximum warping occurred, and (3) final shortening of a companion specimen. The agreement throughout the course of drying between the experimental values for warping and those given by the theoretical equations when these constants are used is excellent for the 1-in. specimen and very good for the 2-in. specimen.

The above shows that if data on warping and data on shortening are analyzed separately, either group of data will appear to be in accord with the theory if the thicknesses of the specimens do not differ too much, but the values of f and k obtained from the two groups of data will be different. The fact that the factor k is an empirical rather than a fundamental property of the material is believed to be the chief reason why both groups of data cannot be represented satisfactorily by one set of constants. The empirical nature of f is considered to be of only secondary importance in this study because it has much less effect than k on the shortening-vs.-time and warping-vs.-time relations.

Effect of differences in k on warping and shortening

The effects of differences in diffusivity on the theoretical values of warping (Equation 23)* and shortening (Equation 22)* are shown in Fig. 17 where these quantities are plotted against the parameter $\sqrt{ft/b}$ for three different relative values of k. As shown, differences in k have practically no effect on the early warping; each curve follows the same course until it approaches its maximum point. The lower k, the greater the maximum warping. This effect on maximum value of warping, of course, would be anticipated because of the effect of k through the parameter fb/k $(= B)$ as shown in Fig. 9.* As shown by the curves for shortening in Fig. 17, the effect of k upon shortening is entirely different from its effect on warping. The rate of shortening is materially reduced by a reduction in k, but the maximum shortening is unaffected.

This theoretical analysis of the different effects of changes in k on shortening and on warping has been useful in explaining differences in performances of concretes made with cements of different composition. For example, experimental results from concretes made with two different cements are shown in Fig. 18. Mix C was used. A comparison of Fig. 17 and 18 leads to the conclusion that the coefficient of diffusivity for concretes made with cement No. 5-1500-1.9 is lower than for con-

*See Part 1.

cretes made with cement No. 1-1500-1.9*. Concretes made with cement from clinker No. 5 shortened at a lower rate but according to data not plotted eventually shortened more than concretes from clinker No. 1.

Before this explanation was found, it seemed surprising that of two groups of specimens drying from one side only, subjected to the same exposure, one group would warp more and shorten less than the other group. In order that one specimen warp less than a second when the two specimens have the same average shrinkage, the distribution of shrinkage in the first specimen would have to be more nearly uniform. For the same surface conditions, a large value of k through the parameter fb/k, tends to make shrinkage more nearly uniform and therefore is accompanied by less warping. An increase in uniformity of shrinkage also reduces the shrinkage stresses in an unrestrained specimen and therefore reduces the tendency for spontaneous cracking. (Fig. 14 Part 1—shows how the theoretical maximum stresses depend on the parameter fb/k $(= B)$.)

Effect of alkali content on k and its possible effects on cracking

It had been observed from various laboratory tests designed to measure cracking tendencies that concretes made with cements from clinker No. 5 tended to crack more than those made with cement from clinker No. 1, even though measurements often showed less volume change at the end of a given period of drying for the concretes of clinker No. 5. This greater cracking tendency of cement from clinker No. 5 was attributed to its higher alkali content, since this appeared to be the only important difference in their chemical compositions. Attempts to evaluate k for concretes made with cements from these two clinkers showed that for the same mix proportions the value of k for concrete made with the cement of higher-alkali content was only one-half that made with the cement of lower-alkali content. These observations suggested the possibility that: alkali reduced k, a reduced k resulted in higher shrinkage stresses, and higher stresses resulted in more cracking.

To investigate this effect of differences in alkali content more fully, several tests were made using cement No. 1-1665-2.48. The procedure was to add 0.91 per cent Na_2O by weight of cement in the form of $NaOH$ to the mixing water of one of two companion mixes. The results of one test using mix B are shown in Fig. 19 where shortening and weight losses of prisms are plotted against period of drying.

The dimensions of the prisms were 2½x2½x11¼ in. They dried from all surfaces except the ends. By using for the specimens containing added alkali a time-scale equal to one-third the scale used for the regular specimens the corresponding curves for both sets of specimens approxi-

*As explained in the Appendix, the first number is the clinker number, the second is the specific surface (Wagner method), and the third is the percentage of SO_3.

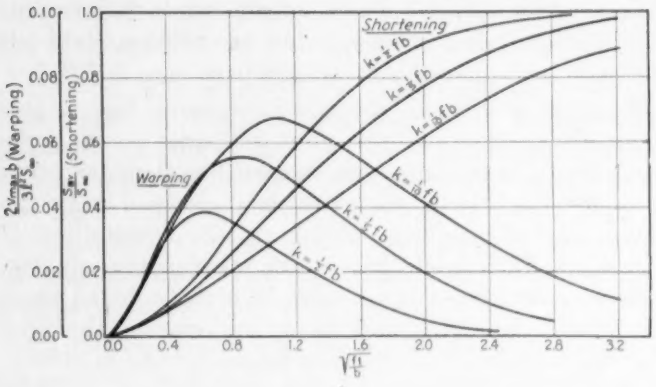


Fig. 17—Theoretical effect of k on the course of contraction and warping of prisms drying from one side only

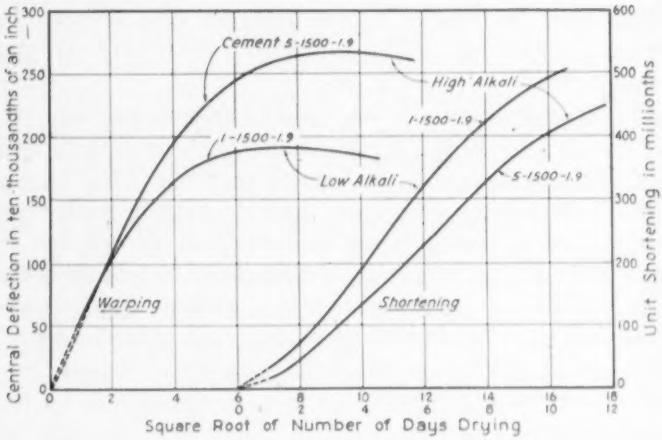


Fig. 18—Warping and shortening of prisms that differ primarily in the alkali content of the cement used

Prisms 3-in. thick in the direction of moisture travel. Deflection measured over a 32-in. span. Mix C. Cured 7 days.

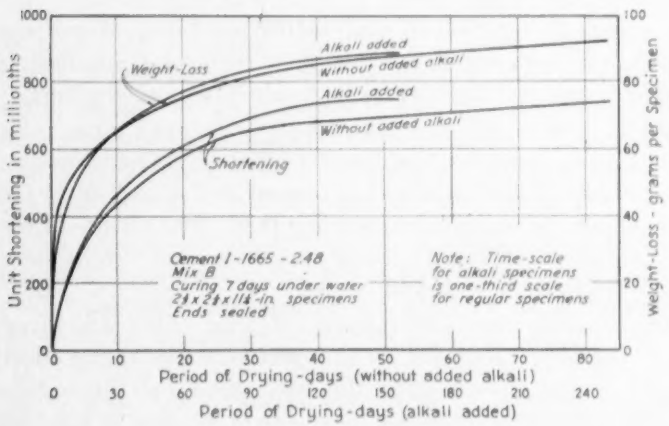


Fig. 19—Shrinkage and weight-loss for specimens with and without added alkali

mately coincided, indicating that the main effect of the added alkali was to reduce the diffusivities for both shrinkage and moisture flow to one-third the value without added alkali.

The effect of added alkali in reducing the diffusivity of shrinkage for cement of clinker No. 1 is in accord with data reported by Haegermann.[21] Haegermann was primarily interested in the effects on shrinkage of additions of various sulfates to cements of different C_3A contents. The sulfates tried were ferrous, calcium, magnesium, sodium, and potassium. The amounts added were such as to increase the SO_3, content 1 per cent, based on the cement. Five cements ranging from 15 per cent computed C_3A content to zero per cent C_3A were investigated.

The data were presented by Haegermann in the form of curves. For each cement, the curves representing the sodium and potassium sulfate additions are of noticeably different shape from the other curves for the same cement, the difference in shape being such as would result from a lower diffusivity. Since Haegermann did not give data on loss in weight during drying, it can only be inferred from the data on shrinkage that the sodium and potassium sulfates also reduced the diffusivity of moisture flow.

From theoretical consideration, it appears that any highly soluble material should reduce the relative rate of drying; i.e., should increase the time required to lose a given percentage of the total amount of moisture to be lost.* However, since many other factors affect the rate of shrinkage, and alkalies have many other effects which may indirectly affect shrinkage, one should expect many real and apparent contradictions to the above indication that an increase in alkali content will retard shrinkage.

The effect of the added alkali on cracking was investigated by means of the "wedge test"† and by the "restrained-shrinkage test" (subsequently described). The result was that specimens of higher alkali content showed a much greater tendency to crack, as measured by these tests.

Other tests made in this laboratory show that for cements containing an appreciable percentage of tricalcium aluminate, an increase in alkali content will increase final shrinkage of laboratory specimens unless the increase in alkali is accompanied by an increase in gypsum. The greater

*This reasoning is based on the supposition that at least part of the flow of water in concrete is by means of the following cycle: evaporation at an air-water interface, vapor diffusion across air space, capillary flow in liquid filled space, and again evaporation at air-water interface. Since the diffusivity of the soluble material within the liquid is finite rather than infinite, at any air-water interface at which water is evaporating, the concentration of soluble material will be higher than that for equilibrium with the adjacent liquid and thereby tend to restrict evaporation at this interface, and at any air-water interface at which water is condensing the concentration of soluble material will be lower than that for equilibrium with the adjacent liquid and thereby tend to restrict condensation at this interface. Therefore any highly soluble material should retard the drying by reducing the diffusivity of moisture flow.

†A specimen is cast in the form of a wedge and, after curing, is permitted to dry from the two non-parallel surfaces.

tendency to crack of the specimens with higher alkali content might have been due, at least in part, to a decrease in diffusivity and an increase in final shrinkage.

On the other hand, the possible benefits from alkali should not be overlooked. The lowered rate of moisture loss will permit the interior of concrete to retain sufficient moisture for additional hydration for a longer time after drying of the surface begins. Prevention of complete drying of the interior during the usual drying season should be especially advantageous in preventing cracking when restraints against shortening are present. Tests in this laboratory have also shown that concretes of higher alkali content have greater capacity for plastic flow, which is a favorable property.

Decrease in k as drying proceeds

As drying proceeds, the value of the coefficient of shrinkage diffusivity k apparently decreases. This decrease no doubt results from a progressive decrease in the apparent diffusivity of moisture, diffusivity of moisture probably being a function of the moisture content. If diffusivity of moisture is a function of moisture content, then the shrinkage diffusivity can be considered to be a function of the shrinkage S and the differential equation becomes non-linear. Adding particular solutions, as was done in Part 1, is then not permissible.

However, if in place of considering k to be a function of the dependent variable S it is considered to be a function of the independent variable t and of the dimensions of the body, then the differential equation remains linear. Furthermore, if the factor f is considered to vary with time in a like manner so that the ratio f/k remains constant (see, for example, Equation 2a), then all of the equations for displacements, stresses and strains developed in Part 1 still apply if the symbol t appearing in them is replaced by a function of t and the dimensions. The changes suggested above amount to a continual change in the time-scale so that the time required for given conditions to develop becomes progressively longer. By modifying the theory in this way better agreement with experimental results can be obtained.

Fig. 20 is an example of applying the foregoing analysis. The plotted points are from experimental data on the average warping of four 3-in. specimens of concrete of mix B with cement 1-2280-1.94. When an attempt was made to select constant values of f, k, and S_∞ to be used in the theoretical equation that would give curves in agreement with all of the experimental values, not all the data could be brought into agreement with the theoretical equation. But by taking the following values for the factors, a better fit was obtained.

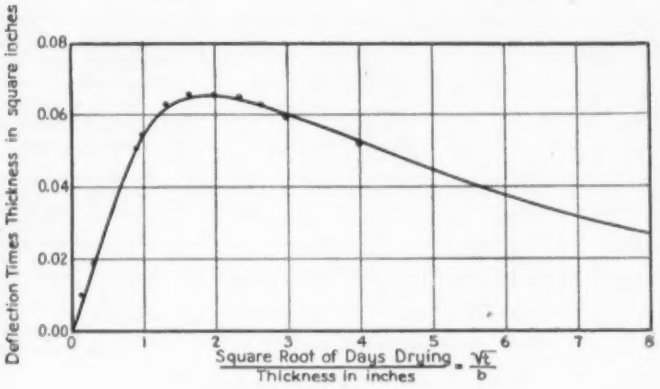


Fig. 20—Comparison of theoretical and experimental warping

Points are from the average warping of four 3-in beams. Span 32-inches. Mix B. Cement 1-2280 – 1.94. Cured 7 days.

$$k = 0.10 \sqrt{\frac{2}{2 + t}} \text{ in}^2\text{/day}$$

$$f = 1.67\ k \text{ in/day, i.e., } \frac{fb}{k} = 5$$

$$S_\infty = 765 \times 10^{-6}$$

When these values are introduced into the differential equations and a solution made, the symbol T in the final equations for warping, etc., is replaced by

$$\frac{4k_o}{b^2}\left[\sqrt{\frac{2 + t}{2}} - 1\right]$$

where k_o is the initial value of k or 0.10 sq. in. per day. For convenience in making computations preliminary to plotting of the theoretical curve, t was expressed in terms of T, or $t = \frac{b^2T}{k_o} + \frac{b^4T^2}{8k_o^2}$. The tabular values used for constructing the curve are given below:

From Table 4		Computed Values		
T	$\frac{2bv_{max}}{3l^2S_\infty}$	t	$\frac{\sqrt{t}}{b}$	$v_{max}b$
0.01	0.0152	1.00	0.333	0.0178
0.03	0.0326	3.61	0.635	0.0384
0.10	0.0538	19.1	1.46	0.0634
0.15	0.0557	36.4	2.01	0.0655
0.20	0.0539	58.5	2.55	0.0635
0.30	0.0470	118	3.62	0.0553
0.50	0.0336	298	5.75	0.0395
0.75	0.0219	639	8.41	0.0258

The better agreement that can be obtained by the modified theory probably would not compensate for the extra work in all cases. Since the

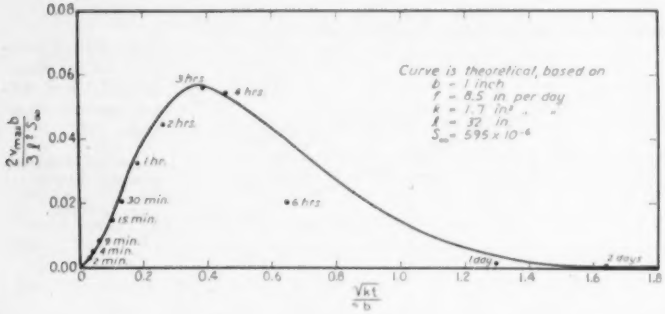


Fig. 21—Comparison of theoretical and experimental warping of beam absorbing moisture from one side

Points represent the warping of a 1-in. beam, having all but one of its surfaces sealed, during submersion in water. Cured 7 days and then dried at 50% R.H. for 10 months before testing. Mix B. Cement 1-2280-1.94. Span 32 inches.

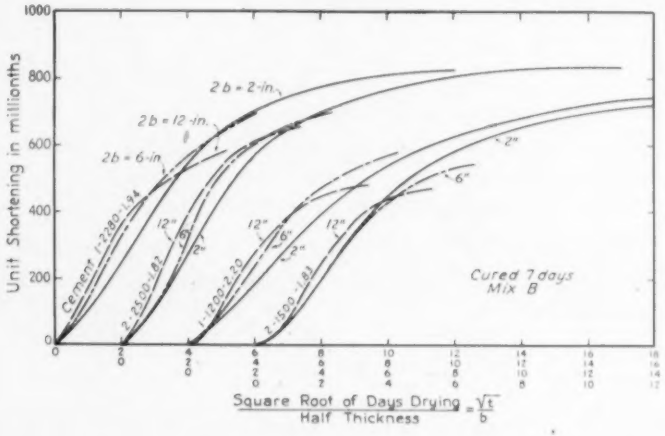


Fig. 22—Shrinkage of slabs of different thicknesses drying from two opposite sides

forms of the resulting differential equations, rather than the reasonableness of the assumption used, were given first consideration, the value of such modification is greatly reduced.

Warping of prisms during absorption of moisture from one side

Expermental results indicate that the theoretical equations are as applicable to the swelling of concrete as they are to the shrinking. However, the factors f and k are different; they are much larger than for shrinking if the exposed surface is submerged in water. Agreement between theory and experiment that is somewhat better in most respects than that usually found in absorption tests in this investigation is shown in Fig. 21. The plotted points give experimental values and the curve gives theoretical values for a 1-in. prism of concrete, mix B with cement 1-2280-1.94. The specimen, which had all surfaces sealed except one, had previously been dried at 50 per cent relative humidity for ten months. This specimen reached a maximum warp after about three hours' absorption of water and at the end of 24 hours had returned to approximately

zero warp where it remained for the rest of the test, a period of one month. The experimental value for the warp after 6 hours' exposure is considered to be in error. Other tests on 1-in. prisms did not show the indicated large decrease in warp between the fourth and sixth hours of exposure.

Although the amount of experimental data on swelling is yet small, the indications are that the application of the theory as developed in Part 1 is limited, first, because at the beginning of wetting the moisture already present will ordinarily not be uniformly distributed; second, because of having remained wet longer, the cement in the interior regions will have hydrated more than that closer to the drying surface; and third, as the concrete becomes wet again, hydration again starts. Agreement with diffusion theory is not expected while hydration is occurring at an appreciable rate, especially if the formation of hydration products causes expansion.

Effect of thickness on rate and amount of shrinkage of walls or slabs

In an investigation of the effect of wall or slab thickness on the rate and amount of shrinkage the results shown in Fig. 22 were obtained. The specimens from which the data were taken were made of mix B. Cements of two different compositions and a fine and a coarse grind of each are represented. The specimens were cured seven days under water. The specimens were 34 inches long and of either 2x6- or 2x12-in. cross section. By sealing all but two surfaces the prisms were made to represent slabs or walls of 2-, 6-, and 12-in. thicknesses drying from two opposite sides. For example, the specimens that represented a wall 12 in. thick were 2x12x34-in. and dried from only the 2x34-in. surfaces, there being 12 inches between these surfaces.

Sets of gage-points were cast in these pseudo slabs so that the shortening over three or four 30-in. parallel gage lines could be measured on each specimen. Details are shown in the Appendix. Each curve was obtained by averaging the results from four specimens of a kind.

As shown in Fig. 22, the results from these concrete specimens are in general similar to those obtained on the cement-silica mix discussed previously (see upper diagram, Fig. 16.). The curves have the characteristic S-shape found for similar plotting of data from smaller specimens. The thicker the slab the greater its fb/k $(=B)$ and, according to theory as shown by Fig. 8 (of Part I), the greater the shortening should be for a given value of the abscissa, $\sqrt{t/b}$. The experimental data are partly in agreement and partly in disagreement with the theory in this regard. In the middle, straight-line portions of the curves, the curves are in the correct positions relative to each other, but in every case the relative positions become reversed at larger values of $\sqrt{t/b}$. Also, the relative

positions of the curves representing the two coarser grinds are reversed at the very beginning portion of the curves. This latter deviation from theory can be explained on the basis of a non-linear stress-flow relationship.

If the positive plastic flow* exceeds the negative, the specimen will not shorten as much as it would if plastic flow did not take place. Of course, if the total positive and negative flows are equal (algebraic average = zero), the length of the specimen is not changed by plastic flow and the algebraic average must be zero in an unrestrained specimen if the stress-flow relationship is linear. However, if the stress-flow relationship of the concrete is non-linear over the range of stresses developed, the flow increasing more rapidly than the first power of the stress, then, as explained in Ref. 20, shortening of the specimen is reduced by plastic flow. The reduction would be more pronounced for the thicker specimens (those of greater b) because, as shown by Curve B of Fig. 14 (Part I), an increase in the parameter fb/k ($=B$) results in an increase in the maximum stresses. The above explanation (on the basis of a non-linear stress-flow relationship) as to why the relative positions of the first parts of the curves representing the two coarser grinds were reversed from what they should be according to theory is not entirely satisfactory because the question arises as to why the curves representing the finer grinds were not reversed also. This point will be mentioned again and an additional explanation given in a later section after other tests with these concretes are reported.

The reversal in relative position of the curves beyond the straight-line portion is attributed to the lesser final shortening of thicker specimens and to the reduction in diffusivity as drying proceeds. The thicker the specimen the less the final shortening because:

(1) The very low rate of drying from the interior of thick specimens is favorable for continued hydration and additional hydration reduces shrinkage tendency.

(2) In a thicker specimen the region losing moisture at an appreciable rate is under greater restraint and for a longer time than in a thinner specimen; consequently, more inelastic elongation is developed.

Plastic flow

Before discussing further the results of measurements of plastic flow, certain common usages of the term will be explained. The term usually calls to mind permanent deformations of the infinitesimal elements of a body as a result of stresses. In many cases, especially if shrinkage-stresses or thermal stresses are present, neither the actual stresses nor the deformations produced by them are known.

*The term "plastic flow" is synonymous with the term "creep" as used by many writers. It is used for either tensile (positive) or compressive (negative) inelastic deformation. See the section on "Plastic Flow" for further explanation of the term.

In the usual measurements of plastic flow the quantity measured is the inelastic deformation of a body that results from applied loads. From these measurements computations are made of the inelastic deformations of the individual elements, i.e., average unit deformation if the load is axial or unit deformation of the outer fiber if the load produces flexure. If stresses from other sources are not present, the computed values may be representative of the actual plastic flow. But if stresses from other sources are present, the computed and actual values may differ appreciably. Therefore, if in addition to load stresses a specimen is under stress as a result of non-uniform temperature or non-uniform shrinkage, it should be made clear whether the term plastic flow refers to the resultant plastic flows of elements or to only computed plastic flows produced by loads. Since the effects of load and the effects of drying are not simply additive, there is no clear basis for deciding how much of the total deformation is due to the stresses arising directly from the load. In agreement with previous writers, the deformations produced by loads will be taken as the difference between the deformations of loaded specimens and the deformations of identical specimens under the same drying conditions but not under load. Only the deformations produced by loads will be computed and represented by curves, but in the *interpretation* of results consideration will be given to what the *actual* inelastic deformations are believed to be.

As shown by the formulas for plastic flow used in this paper, the total deformation produced by load is divided into two parts, elastic and inelastic. The elastic part is considered to be that which would be recovered immediately if the load were removed; it is determined from the computed load-stresses and the "dynamic" modulus of elasticity. The remaining part is considered to be the plastic flow produced by the load.

Some investigators make a slightly different division in that the elastic deformation is considered to be that which was produced immediately upon application of the load rather than that which would be recovered immediately upon removal of the load. The two values are equal if the modulus of elasticity does not change during the test. Some writers prefer to divide the total deformation produced by load into three parts: (1) that recovered immediately upon removal of load, (2) that not immediately but eventually recovered, (3) the permanent deformation. McHenry[22] restricts the use of the term plastic flow to the third part. This division into three parts has merit, especially for those cases in which the second part is an appreciable percentage of the total. For the data given in this paper no separation of the second and third parts could be made, but the permanent deformation (3) is believed to be much greater than the temporary (2).

If the specimen is not shrinking while it is under load, then a considerable part of the inelastic deformation is probably only temporary and apparently the result of viscous flow in the adsorbed water films. After removal of the load, the elastic constituents of the gel-structure tend to restore the original shape but are retarded by the viscosity of the adsorbed water films. However, if an elemental volume* of cement paste is shrinking while under stress, the conditions are different. The loss of moisture introduces relatively large interparticle forces which tend to change the relative positions of the colloidal particles within an element. Some adjacent particles are pulled closer together but others are moved further apart. During this time of movement the directions of relative motion of the particles may be appreciably affected by stresses on the element. In this way stresses on an element during the time it is shrinking may produce comparatively large permanent deformations.

The foregoing is one explanation for the much larger amount of plastic flow that a load will produce on a drying specimen compared to what it would produce if either the specimen were prevented from drying or had previously been dried. It is also an explanation of the relatively great capacity for the concrete near the drying surface to deform plastically without cracking. If the analysis is correct, then a definite stress-flow relationship cannot be ascribed to a given element of concrete since the amount of plastic flow would depend not only upon the magnitude and duration of stress on an element but also upon the changes in moisture content that occurred while the element was under stress.

Because most of the inelastic deformation was considered to be permanent and because the increase in deformation with time was considered to be controlled chiefly by changes in distribution of shrinkage and shrinkage-stresses with relatively small lag in time after the development of shrinkage-stress, the term "plastic flow" rather than "creep" was selected for the inelastic part of the deformation.

It is important to know in what way the pastic flows of the individual elements contribute to the inelastic deformations of the body as a whole. For example, if a body is under axial load the plastic flow in tension or compression caused by the load is the difference in the *algebraic sum* of the inelastic deformation of each element and what the algebraic sum would have been if the body had not been under load. But the plastic flow of a body under flexural load depends on the *moment* of the inelastic deformation of each element with respect to the "neutral axis." Both plastic elongation on the tension side and plastic compression on the compression side of a beam under flexural load contribute to the measured plastic flow of the beam as a whole.

*In this discussion non-homogeneity of the cement paste is recognized and an element of paste is not infinitesimal but large enough to be essentially like adjacent elements.

Summary of remarks on plastic flow. The actual plastic deformation of elemental volumes of a specimen may be much different from that computed on the basis of laboratory experiments if shrinkage-stresses are present, but in this paper the plotted curves represent such computed values. Computed values are based upon the difference in the deformations of loaded and not loaded specimens. The term "plastic flow" is used in this paper to refer to either actual or computed plastic deformation. Plastic deformation is arbitrarily defined as that part of the total deformation produced by stress (either by actual stress or by load-stress as indicated by the text) that would not be immediately recovered upon removal of the stress.

Effect of thickness on stresses and plastic flow when the slab is partially restrained against shortening

Companion specimens of the same size and sealed in the same manner as those represented in Fig. 22 were partially restrained against shrinkage by specially designed steel bars, somewhat as were those described by Carlson.[23] The main features of the steel bars are shown in Fig. 23. (The concrete specimen illustrated in Fig. 23, however, is from another test in which the concrete was allowed to dry from all sides and only one bar was used per specimen). Each specimen of 2x6-in. cross section contained two $\frac{5}{8}$-in. diameter bars, and each specimen of 2x12-in. cross section contained four $\frac{5}{8}$-in. diameter bars. The arrangement of bars is shown in the Appendix, and in Fig. 24.

A rubber tube covered the central 20 inches of each bar so as to prevent bond over a 20-in. gage length, thereby insuring the same axial force in the bar over all sections of the gage length. That part of each steel bar not covered with rubber was threaded and thus the bars were anchored to the concrete for a distance of 7 in. on each side of the gage length. Because of this anchorage the shortening of the steel bar over the gage length is equal to the shortening of the concrete over the same gage length. Moreover, as is obvious from considerations of equilibrium, the force in the concrete in this gage length is equal and opposite to the force in the steel in the same gage length. Therefore, the average unit stress in the concrete can be computed from the change in length, modulus of elasticity, and percentage of steel. The formula is

$$\sigma_c = \frac{A_s E_s}{A_c} \frac{\Delta l}{l}$$

where σ_c is average stress in the concrete
A_s is cross-sectional area of the steel
A_c is cross-sectional area of the concrete
E_s is Young's modulus for the steel

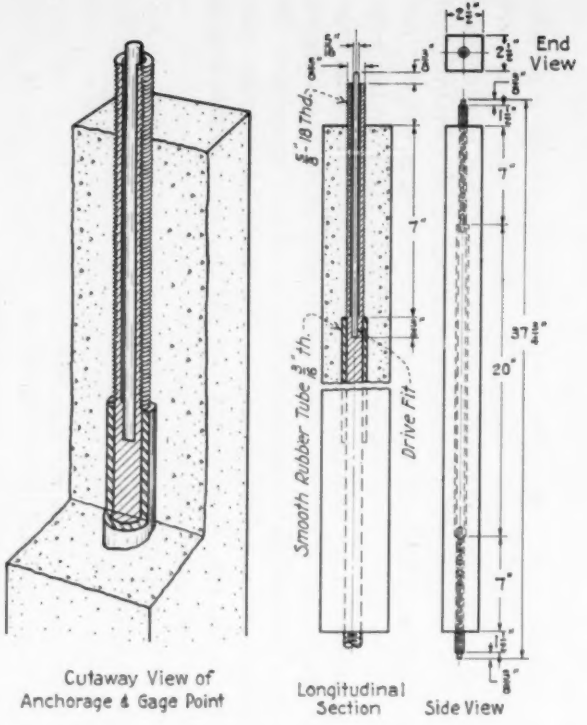
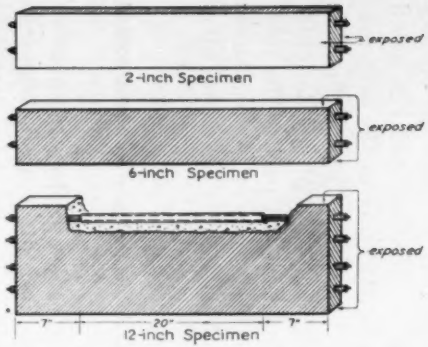


Fig. 23 (above)—Details of cracking—test specimen



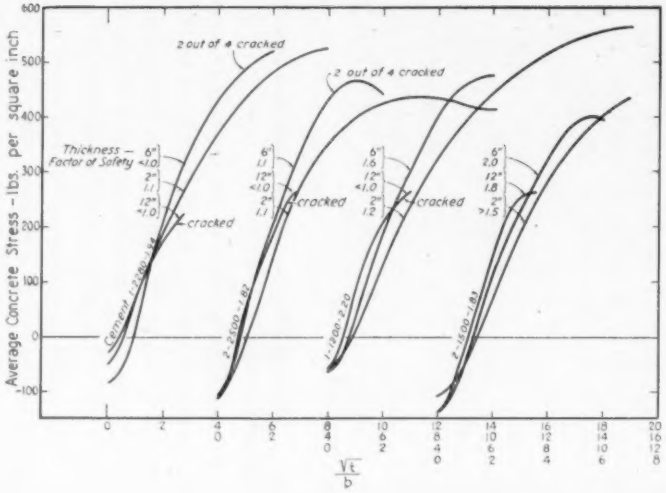


Fig. 24 (right)—Average stress in partially restrained specimens (each curve the average of four)

Δl is net change in length over gage length after corrections have been made for any change in temperature.* Δl is negative if the specimen has shortened.

l is gage length.

Not only the average stress caused by the restraint but also plastic flow caused by this stress can be computed if the modulus of elasticity of the concrete is known and the assumption is made that the shrinkage tendencies of the restrained specimens are the same as those for companion unrestrained specimens of the same size. The formula is

$$c = S_{av} - \frac{\sigma_c}{E_c} + \frac{\Delta l}{l}$$

where c is unit plastic flow caused by restraint,

S_{av} is the unit shortening of the free-shrinkage specimens, and

E_c is Young's modulus for the concrete.

Performance of partially restrained specimens. As explained in the Appendix all specimens were cured under water. The specimens tended to expand during this storage and consequently the concrete in those partially restrained with restraining bars was compressed. Therefore, for a short time after drying began, the direction of the plastic flow produced by the restraining bars was in a negative direction. Shortly after drying began, the stresses in the restraining bars changed from tensile to compressive, and the average stress in the concrete changed from compressive to tensile.

Under the conditions of this test the average stress reaches a maximum and then slowly decreases if failure by spontaneous cracking does not occur. A specimen's average stress and its shortening necessarily reach their maximums simultaneously if the temperature remains constant. Therefore, the time of maximum average stress is the time when the rate of average shrinkage equals the rate of plastic deformation. During the decrease of average stress, the rate of plastic deformation exceeds the rate of shrinking.

Ordinarily in this test the specimens are not permitted to reach a final equilibrium state in regard to shrinkage, shrinkage-stress, and plastic flow. But just after the maximum restraining force has been developed additional tensile load sufficient to cause failure of the specimen is applied. This load is applied to the protruding threaded ends of the restraining bars by a machine designed for the purpose. While the load is being applied, measurements are taken so that the added stress in the concrete can be determined. Further details are given in Fig. 33 of the Appendix.

*All tests were conducted in a room maintained at 76 ± 1°F and a relative humidity of 50 ± 2%, except for occasional deviations from these limits.

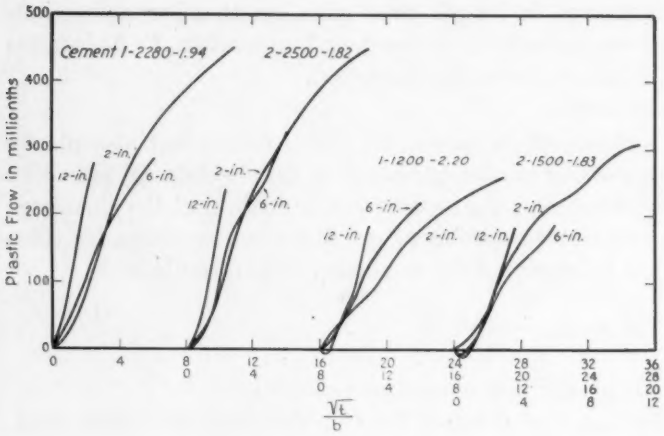


Fig. 25—Plastic flow in the partially restrained specimens of Fig. 24

Factor of safety. The purpose of the testing just described is to learn how close the specimen comes to cracking spontaneously. The ratio of the computed stress at failure to the maximum average shrinkage-stress is called a factor of safety. Specimens that crack spontaneously are reported as having a factor of safety less than unity. Results showing computed average shrinkage-stresses and plastic deformation are shown in Fig. 24 and 25, respectively. The record of the number of specimens of each cement that cracked spontaneously and of the average factors of safety (f.s.) of those that did not crack is also shown in Fig. 24.

Effect of thickness on plastic flow. Attention is called to the similarity of the three sets of curves in Fig. 22, 24, and 25. The similarity is not to be interpreted as necessarily indicating that the plastic deformation of an element is proportional to its stress. One is tempted to make this interpretation because, if it were true, then the plastic flow of a specimen would depend only on average stress and not on the distribution of stress. In general, the diagrams show that the 12-in. specimens developed considerably more plastic flow for the same amount of average stress than either the 2- or the 6-in. specimens.

There are several possible reasons for this: first, because of the lower rate of shortening, the larger specimens will have been under a given range of stress longer than the smaller specimens and therefore would be expected to have more plastic flow for the same stress. Since the time required for the same amount of shortening is approximately proportional to the square of the thickness, the 12-in. specimens will in general have been under a given range of stress about four times as long as the 6-in. specimens. Second, since the thicker specimens will have higher maximum stresses, the additional plastic flow could be accounted for by a non-linear stress-flow relationship whether or not this relationship

for each element was modified while the element was losing moisture rapidly. Third, the assumption that shrinkage tendencies of the restrained-shrinkage and free-shrinkage specimens are equal is not entirely correct and consequently their computed plastic flow are in error. A difference in the shrinkage tendencies of the 12-in. free and restrained specimens might result since the arrangement of the four bars was such as partially to obstruct the flow of moisture.

Probably all factors listed above contributed to the results. Of the factors causing the computed plastic flow to be greater in the 12-in. specimens, the author is of the opinion that the non-linear stress-flow relation contributed much more than the difference in duration of given stresses.* That the maximum stresses in the larger specimens are higher is shown by the fact that all the 12-in. restrained specimens of three of the four cements cracked (see Fig. 24) whereas only a few of the 6-in. specimens and none of the 2-in. specimens cracked.

According to most of these arguments the 2-in. specimens should have less plastic flow than the 6-in. specimens, whereas in general they have slightly more for the same shortening and for the same average stress. A complete explanation for this is not at hand, but the lesser extent of hydration of the cement in the 2-in. specimens because of their more rapid drying may be a factor. Also, the exposed surfaces of the 2-in. specimens were the top and bottom surfaces as cast, whereas the drying surfaces for all the other specimens were the sides as cast. Bleeding and settlement of the plastic mix before initial hardening is complete always makes the concrete near the top and that near the bottom as cast different from that at the sides. It must also be remembered that the computed plastic deformation may be more or less than the real plastic deformation of the material.

Use of beams drying from only one side for determining probable stresses in slabs or walls drying from two opposite sides

To obtain information on plastic flow and on the magnitude and distribution of stresses in unrestrained walls or slabs drying from two opposite sides another set of specimens, also companion to those represented in Fig. 22, were made. These specimens differed from those of Fig. 22 in that they were permitted to dry from only one side instead of two opposite sides and in that the thicknesses of corresponding specimens were just half those of Fig. 22. Since they were half as thick and dried from only one side instead of two sides (see Appendix), any one of these specimens was considered to have the same conditions of drying and consequently the same distribution of shrinkage tendency as either half of a corresponding specimen represented in Fig. 22.

*Most contemporary writers on the inelastic properties of concrete apparently would take the opposite view. This difference in viewpoint is explained and an argument for the author's view is given in Ref. 20.

If the distribution of shrinkage tendency is the same and if the correct external forces are applied so as to make all the deformations the same as those of either half of the corresponding specimen, then the distribution of stresses will also be the same.

Eight specimens of a kind were made, four of which were allowed to warp freely and four were restrained against warping. It was not feasible to distribute the external restraining forces on the specimens to be restrained against warping in exactly the manner that the mutual forces between the two halves of the corresponding specimens were distributed. As shown in the Appendix, the method adopted was to support the specimen as a simple beam and to apply enough force at the quarter-points to prevent warping of the central half.

As discussed previously and as shown by Equation 19*, the amount of warping of a specimen free to warp is indicative of the non-uniformity of shrinkage tendency. From similar considerations it follows that the amount of moment necessary to prevent warping is indicative of the non-uniformity of stresses resulting from the non-uniformity of shrinkage tendency. Furthermore, the difference between the actual moment required to prevent warping and that computed from the amount of free warping, assuming no plastic flow, gives an indication of the distribution of plastic flow. The results for one of the four cements are shown in Fig. 26. The lower curves give the actual moment developed, M, divided by the section modulus, I/c, for beams of three different thicknesses. The upper curves show what the Mc/I would have been if the restraining moment had not produced plastic flow. The upper curves are obtained from the measured values of the warping tendency and Young's modulus of companion specimens. Young's modulus was determined from resonant frequency of vibration.

Stresses based on flexure formula. Since the ordinates in Fig. 26 are in terms of Mc/I, they represent the stress in the outer fiber according to the elementary flexure formula. According to the lower curves of Fig. 26 the computed stresses for the 1-in. specimens reached a maximum of 410 lb. per sq. in. by the end of the first day of drying. The computed maximum stress in the 3-in. specimen was 490 lb. per sq. in. and was reached in 14 days. The computed stress in the 6-in. specimens had reached 500 lb/in^2 after 36 days and the indications are that, had the test been continued, the computed stress would have reached a maximum of about 550 lb per sq. in. after about 100 days of drying. These computed stresses in the outer fiber, based as they are on the flexure formulas, are of course not the actual stresses. The actual stresses in the drying surface will build up very rapidly (see curve $y/b = 1.0$ in Fig. 12*) and

*See Part 1.

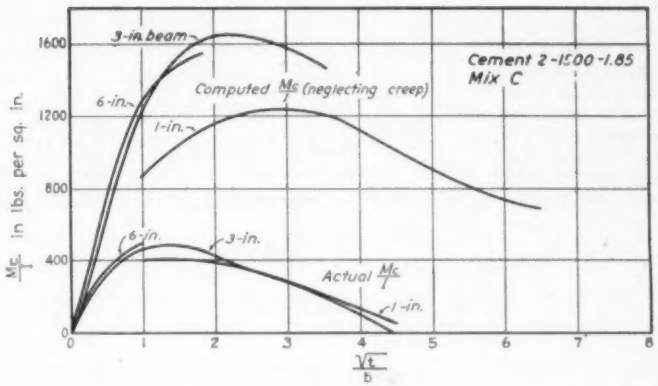


Fig. 26—Comparison of actual moment necessary to keep beams from warping with the conputed moment necessary to straighten companion beams that are free to warp. (Plotted in terms of $\frac{Mc}{I}$)

will build up as rapidly in the thicker slabs as in the thinner. The actual stresses are probably better represented by the solid curves of Fig. 27.

Stresses based on modified theory. The solid curves of Fig. 27 show stresses based upon a modified theory. These curves were obtained by substituting appropriate values of the parameters y/b, kt/b^2, fb/k, E, and S_∞ into Equation 20 of Part 1. For the construction of these curves the theory as presented in Part 1 was modified in that, instead of using constant values for the factors k, f, S_∞, and E, the following procedure was pursued:

(1) The ultimate shrinkage S_∞ was set equal to 750, 700, and 600 millionths, respectively, for the 1-, 3-, and 6-in. thick specimens. The selection of these separate values rather than one value for all specimens was governed by the apparent ultimate unit shortenings of the corresponding free-shrinkage specimens (Fig. 22).

(2) The ratio f/k was set equal to 2.5 in.$^{-1}$, i.e., fb/k was 2.5 for 1-in., 7.5 for 3-in. and 15.0 for 6-in. specimens. When this value of f/k and the above values of S_∞ were used, the theoretical maximum values of warping as given by curve A of Fig. 14* were found to be in agreement with the experimental values of maximum warping for each of the three thicknesses of specimens.

(3) A value of kt/b^2 was selected for each period of drying (1, 7, and 28 days) and for each thickness of specimen, such that when substituted along with the above values of S_∞ and f/k in Equation 23* for warping the result would be in agreement with the experimentally determined values for these periods and these thicknesses.

(4) A value of E was selected for each period of drying and each thickness of specimen such that the theoretical moment given by Equation 27 would be in agreement with the experimentally determined values.

*See Part 1.

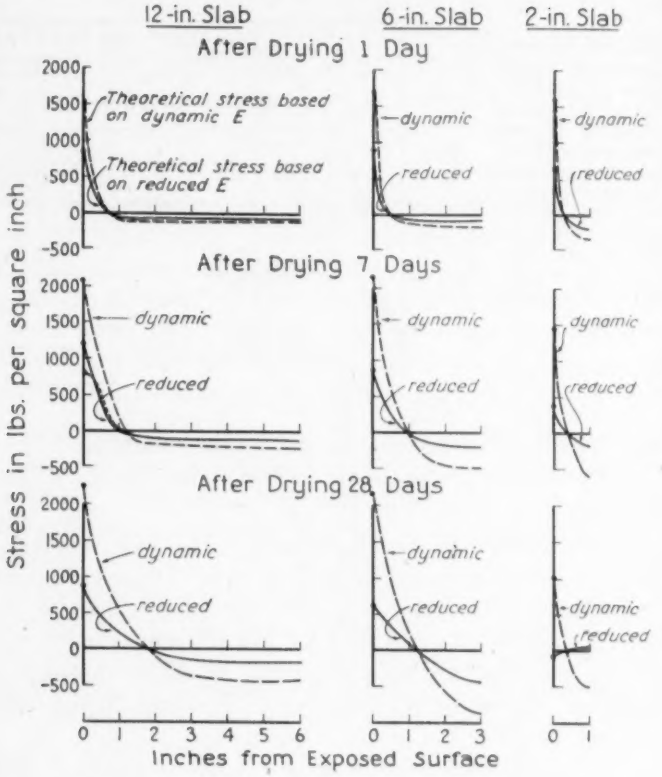


Fig. 27—Theoretical distribution of shrinkage stresses (modified theory) in slabs drying from two opposite surfaces for various drying periods and slab thicknesses

(5) The above values of S_∞, kt/b^2, fb/k, and E together with appropriate values of y/b, were substituted into Equation 20* and computations made for stresses. The solid curves were plotted from these computed stresses.

The dashed curves show the stresses that are obtained when the foregoing procedure of computing stresses is used except that the dynamic modulus of elasticity is used for E. The difference in the curves is a measure of the relief of stress by plastic flow and thus is a measure of the amount of plastic deformation that has occurred.

Although it is believed that for the most part the magnitude and distribution of stresses after the various periods of drying and for the various thicknesses of specimens are about as given by the solid curves of Fig. 27, the curves are probably in error in certain respects. The principal source of error lies in the assumption that the effects of plastic flow can be taken into account by using a reduced modulus of elasticity as is done when Equation 20 is used. Inelastic deformation is cumulative and depends on the past stress-history, not necessarily on the

*See Part 1.

stress at the moment. Furthermore, as explained in the section on plastic flow, the rate of flow for an element will depend on the rate at which the element is tending to shrink and may not be proportional to the stress on the element. Therefore, near the drying surface, where the stress has been relatively high from the beginning of drying, the plastic flow will be greater and the stress will be less than that indicated. Slightly farther inward where the stress has only recently changed from compression to tension the resultant plastic deformation will be less and the stress more than that indicated. The dotted curve in the one diagram of Fig. 27 represents an attempt to show a better estimate of the actual stress.

Reversal of stress by plastic flow. Fig. 26 indicates that eventually the stress in the outer fiber will become negative, i.e., compressive. In all restrained-warping tests that were continued until equilibrium of moisture content was nearly reached, the moment required to prevent warping decreased to zero and would have then become negative if restraint against negative warping had been provided. This means that when a wall dries from two opposite sides or a prism dries from all four sides, eventually the outer shell will be in compression and the inner core will be in tension.

Of interest in this connection is the fact that specimens of neat cement bars have been known to break spontaneously and audibly while resting in place in a storage rack. The explanation is that during the early part of drying large tensile stresses developed in the outer shell. As a result the outer shell was first permanently elongated and then caused to fail in tension, i.e., to crack. As drying proceeded inward, the inner core, which had not yet been stretched, tended to become shorter than the outer shell. The cracks closed, compression developed in the outer shell, and tension developed in the inner core. In some cases this tension was sufficient to cause failure of the core. A specimen would break spontaneously when failure of the inner core occurred at a section where the outer shell was already cracked.

Investigation of properties of concrete by means of slabs or prisms drying from one side only

As the foregoing has indicated, results from prisms drying from only one side have been very valuable for ascertaining in what ways the theory of diffusion is applicable to shrinkage of concrete. They are also valuable for investigating certain properties of concrete, especially if used in connection with the diffusion theory. The chief advantage of drying a prism from only one side is that it tends to warp as well as shorten as it dries and thereby makes possible measurements not obtainable on prisms drying from all surfaces.

As explained previously, the results from prisms drying from only one side indicated a cause for concretes of higher alkali content to show a greater tendency to crack under some conditions. Additional results in regard to fineness of grinding and percentage of gypsum will now be reported for the information they give to illustrate how such prisms may be used to investigate properties of concrete.

Effect of finer grinding on plastic flow. As explained previously, in addition to the one cement represented in Fig. 26 and 27, three other cements were tested at the same time and in the same manner. The results for the other cements were similar in most respects to those shown in Fig. 26 and 27 for the one cement. There were some differences, however. More plastic flow occurred in the specimens made with the two finer-ground cements. The differences in plastic flow with fineness of grinding in this test were similar to and in agreement with the differences shown for these cements in the restrained-shrinkage test (Fig. 25).

The effect of finer grinding on strength is similar to that produced by longer curing and therefore since longer curing decreases plastic flow we might expect that finer grinding would also decrease plastic flow. However, several cements tested in this way all showed that finer grinding increased plastic flow for both restrained-shrinkage and restrained-warping specimens. Other tests in this laboratory indicate that this effect of finer grinding on plastic flow is indirect. A given quantity of gypsum added at the time of grinding is less effective in retarding the early chemical reactions the finer the cement.[24] Lack of proper retardation of the early reactions because of insufficient gypsum results in a concrete of greater shrinkage tendency and greater capacity for plastic flow. Therefore, according to the indications, finer grinding, if unaccompanied by increase in percentage of gypsum, indirectly produces a concrete of greater tendency to deform inelastically.

This still leaves unanswered the question introduced in the discussion of Fig. 22 as to why the effects of plastic flow did not also reverse the relative positions of the curves for the finer-ground cements. Possibly at the very beginning of drying the coarser-ground cements, because of their relatively low strength, flow more readily but become less plastic than the finer-ground cements at the later ages when the strengths are more nearly equalized. However, since only slight differences in the way the different concretes deviate from the theory could account for the results and since concrete deviates from the theory in many different ways there are other possible answers to the question.

One possible answer is suggested in Fig. 24, according to which coarser grinding resulted in greater negative stresses at the end of the curing period. These negative stresses were produced by the tendency of the

concrete to expand during curing, especially the tendency to expand after some resistance to plastic flow had developed. Any tendency for the interior of the free-shrinkage specimens to continue expansion after the surface begins to dry would reduce the rate of shortening at the beginning of the drying period. If, as seems quite probable, this tendency is greatest for the thicker specimens with the coarser-ground cements, then these specimens would shorten relatively less at the beginning of drying than would be indicated by theory.

Effect of added gypsum. The effect of the gypsum in the cement on the properties of the hardened concrete was observed in an investigation in which 21 cements were made from the five clinker compositions listed in the Appendix. By blending various grinds of these clinkers, cements of different finenesses and different gypsum contents were obtained from each clinker. Concretes (Mix C) made from these 21 cements were tested in the manner indicated previously for prisms drying from only one side. However, in this investigation only the 3-in. size of specimen was used.

Where the C_3A content of the clinker was moderate or relatively high, an increase in SO_3, content decreased shrinkage and warping and also decreased plastic flow. Where the C_3A content was low, an increase in SO_3 had relatively little effect. According to other data obtained in this laboratory, a still further increase in SO_3 would have increased the shrinkage.[24] Representative results for a cement of high C_3A content are shown in Fig. 28, 29, and 30. As shown by Fig. 28, the maximum warp of those specimens free to warp was reduced appreciably by increase in per cent of SO_3. The reduction in shortening with increase in SO_3 agrees with that reported previously by other investigators[21, 24, 25].

Fig. 29 shows that the restraint developed by the specimens restrained against warping was in general less with the higher percentages of SO_3. However, increasing the SO_3 from 1.5 to 2.4 per cent had only a very small effect on the amount of restraint developed in the restrained specimens compared to the effect on the warping of unrestrained specimens. The explanation is that although the increase in SO_3 reduced warping it also reduced the tendency to yield under stress. The net result is some reduction in stress but not as much as would be anticipated from the results of the free-warping specimens.

Fig. 30 shows the effect of SO_3 on the factor of safety against cracking as determined by this test of a cement of high C_3A.

SUMMARY AND CONCLUSIONS

The theory that shrinkage of concrete follows the laws of diffusion similar to those followed by the flow of heat is tested by means of specially

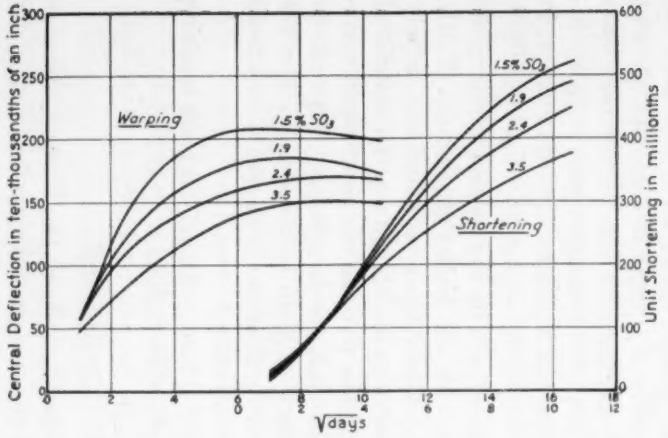


Fig. 28—Effect of gypsum on warping and shortening for a cement of high C_3A content

Cement from Clinker No. 1. Specific Surface 2100. Specimens 2 x 3 x 34-in. Cured 7 days. Sealed on all but one 2 x 34-in. surface. Central deflection over a 32-in. span.

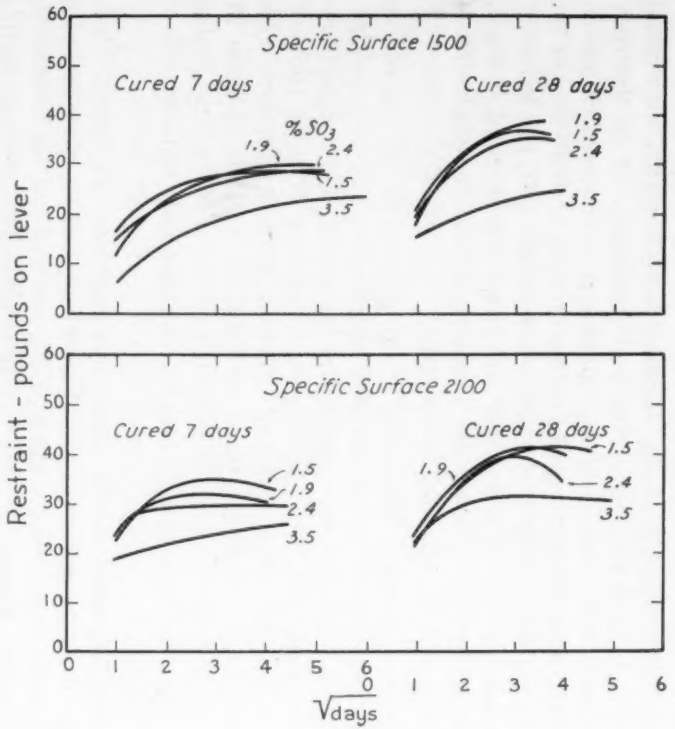


Fig. 29 (right)—Effect of gypsum, period of curing, and fineness of grinding on restraint necessary to prevent warping

Cement from Clinker No. 1. Mix C. 3-in. beams $\frac{Mc}{I} = \frac{32}{3}(W + 4)$ where W is pounds on lever.

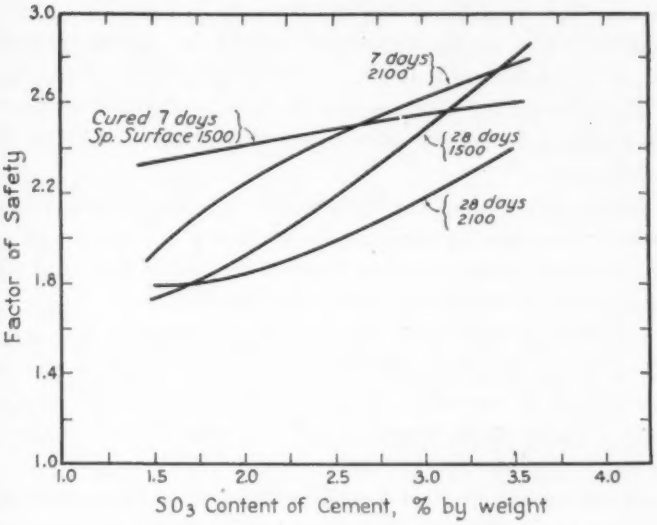


Fig. 30—Effect of gypsum, period of curing, and fineness of grinding on factor of safety against cracking

Specimens same as those represented in Fig. 29.

designed experiments. According to the theory that was developed in Part 1, the shrinking and development of stress in a given concrete under given conditions of drying is considered to be characterized by certain constants. These constants are diffusivity of shrinkage, surface factor, ultimate shrinkage, and Young's modulus of elasticity. Equations were derived in Part 1 giving shortening and warping of prisms versus period of drying in terms of these constants and the dimensions of the prisms.

In Part 2 it is shown that these constants can be selected so that the shortening of a prism as computed by the theoretical equations is in good agreement with experimental values of shortening. Furthermore, it is shown that by using the same constants the shortening versus period of drying of other prisms differing in size and number of sides exposed to drying can be predicted with fair accuracy if the difference in size is not too great. However, it is shown that the theory must be modified to take into account inelastic deformation and to permit the supposed constants to vary with moisture content and size of specimen if the theory is to be in agreement with all results on all types of specimens of a given concrete.

The theory is used to explain various things about concrete; in fact, paradoxically, it is used to explain some of the ways in which concrete does not perform as predicted by the theory. The tendency of large specimens to crack more and shrink less than smaller specimens and the effect of alkali content of the cement in increasing the tendency to warp while reducing the rate of shrinkage are explained on the basis of the

theory. It is shown that when a saturated specimen is dried at 50 per cent relative humidity the stress developed would be much greater than the strength of the concrete if it were not for the effects of plastic flow. It is further shown that when a specimen is restrained against deforming the restraining forces are much less than they would be if plastic flow did not occur.

An example is given of the use of the theoretical equation in determining the distribution of stresses at various times during the drying of a specimen. In this example, consideration is given to plastic flow and to the decrease in diffusivity of shrinkage as the specimen dries.

The restrained-shrinkage test and the restrained-warping test are used to determine a factor of safety against cracking for concrete under conditions of drying and of restraint comparable to those under which the tests are made. These tests, together with tests on free-shrinkage and free-warping specimens, are used to measure plastic flow.

The various tests described in Part 2 and in the Appendix to Part 2 when used in conjunction with the theory given in Part 1 provide a means for studying some of the more fundamental properties of concrete and for predicting the performance of concrete under some conditions in the field.

ACKNOWLEDGMENTS

This paper developed during a study of the causes and the control of cracking of concrete. The author is indebted to many present and past workers in this field. E. A. Ripperger, now Lt. (j.g.) U.S.N. with the Pacific fleet, was responsible for a large part of the experimental work reported herein. He designed (or adapted from earlier designs) most of the special equipment used and was engaged in certain phases of the study previous to the author's participation. All of the work was done under the supervision of F. R. McMillan, Director of Research, and T. C. Powers, in charge of Basic Research.

The author is particularly indebted to Mr. Powers for assistance in preparing the manuscripts. His suggestions in regard to presentation of material and the wording of various paragraphs have been invaluable.

The author also wishes to thank Miss Adele Scott for preparing the diagrams and Miss Virginia Atherton for proofreading the manuscript.

APPENDIX TO PART 2

Mix proportions

Materials	Parts by Weight		
	Mix A	Mix B	Mix C
Water, net	0.5	0.355 to 0.388	0.487
Water added for absorption	...	0.048	0.083
Total water	0.5	0.403 to 0.436	0.570
Cement	1.0	1.0	1.0
Pulverized silica	0.6	....	....
Elgin sand	...	1.28	2.43
Elgin gravel (¾" max. size)	...	1.82	2.97

Consistency

Consistency of Mix B with different cements was maintained fairly constant at from 5 to 6 in. of slump with a 12-in. cone by varying the amount of mixing water. The consistencies of Mixes A and C were allowed to vary with the different cements. Mix C usually gave a slump of from 2 to 4 inches, but with some cements the slump was as little as 1.5 inches and with others as much as 6 inches.

Materials

Cements: One cement designated M was a mixture of four brands of Type I cement, purchased in Chicago. Its specific surface by the Wagner method was 1665 sq. cm. per g. The other cements were prepared from five different commercial clinkers. From each of these clinkers cements of three different finenesses, coarse, medium, and fine, were prepared by grinding at the plant. In addition, two cements, one of low and one of high gypsum content, were prepared from each clinker by grinding in a small laboratory ball mill. The purpose in preparing these five different cements from each clinker was to make it possible to obtain any desired fineness and gypsum content by blending different grinds of the same clinker. In referring to these cements in the text the first number in the designation is the clinker number, the second is the Wagner specific surface, and the third is the per cent SO_3 content by weight.

The chemical compositions of the five clinkers and of cement M are shown below.

Oxides	Cement Clinker No.					Cement M
	1	2	3	4	5	
	Chemical Analysis, per cent by wt. (corr. for minor components)					
SiO_2	21.54	20.67	23.05	27.82	22.56	21.25
Al_2O_3	6.52	5.48	4.14	1.93	5.00	5.98
Fe_2O_3	1.56	2.50	4.35	1.87	2.48	2.69
Combined CaO	64.32	65.00	64.28	65.38	64.06	62.56
MgO	2.17	1.31	1.36	1.75	3.35	3.04
SO_3	0.41	0.19	0.03	0.17	0.20	1.75
Loss on Ign.	0.15	0.85	1.05	0.26	0.37	1.13
Free CaO	0.98	2.71	0.73	0.23	nil	0.79
Na_2O	0.17	0.30	0.05	0.05	1.13	0.28
K_2O	0.16	0.40	0.17	0.22	0.44	0.63
Compounds	Computed Compound Composition per cent by wt.					
C_3S	50.73	66.57	52.37	38.58	51.61	44.15
C_2S	23.49	9.05	26.58	50.66	25.75	27.62
C_3A	14.72	10.29	3.61	1.95	9.06	11.30
C_4AF	4.75	7.61	13.24	5.69	7.55	8.19

Aggregate: Sand and gravel were from Elgin, Illinois. The gravel was screened to pass a ⅜-in. sieve and be retained on a No. 4 sieve.

The sand was graded as follows:

Sieve No.	Per Cent Passing
100	5
48	15
28	43
14	67
8	82
4	100

Pulverized Silica: The silica was from the same source as standard Ottawa sand, but ground to cement fineness. Its specific surface by the air-permeability method was 3200 sq. cm. per g.

Procedure

With some exceptions the procedure in preparing and testing the specimens was as follows:

Preparing the specimens: The materials were mixed in a power-driven open-tub mixer. The fractions of the various-sized aggregates and grinds of cement were weighed and placed in the tub. The mixing schedule was: mix ½ minute dry, 2 minutes wet, rest 3 minutes, mix 2 minutes. (Special tests showed that the grinds of cement were adequately blended in ½ minute of dry mixing in the presence of sand.)

The freshly mixed concrete was placed in steel molds and consolidated by light vibration by placing molds on a platform-type vibrator. Covers were fastened on the molds but not made water-tight. Each mold was equipped with restraining bars, gage inserts, etc., the details of assembly depending upon the tests to be made.

The molds and contents were stored under water at 74 F for one day. The molds were then stripped and the specimens returned to water at 76 ± 1 F in a covered tank where they were left for one hour. The specimens were then removed one at a time, dried with a cloth, and all initial measurements of length, deflection, and weight were made. All dimension measurements were made within 30 seconds after removal from water, and weighings were made as soon thereafter as practical. The specimens were then returned to the 76 F curing tank.

One day before the end of the curing period the specimens were removed for sealing of certain surfaces against the loss of moisture. The surfaces to be sealed were wiped with a cloth and then allowed to air-dry until the surface *just* changed color. During this time, scheduled measurements were usually made. After the color change and before any appreciable loss of moisture by evaporation, one coat of black, quick-drying brushing lacquer was applied. After the lacquer had dried a few minutes, the excess was removed with a cloth and one coat of hot paraffin was applied to the lacquered surface. While a second coat of paraffin was being applied, one thickness of newspaper of appropriate shape was pressed into the still soft paraffin somewhat in the manner in which a paper hanger applies wallpaper. Next, a final heavy coat of paraffin was applied. The layer of paper helped to eliminate pin holes. (When only the ends of prisms were sealed, the paper and final coat were omitted.)

After the specified surfaces were sealed, the unsealed surfaces and exposed steel parts were cleaned, vaseline was applied to the steel parts, and the specimen was returned to the curing tank for an additional day of curing. At the end of the curing period the specimens were transferred from the curing tank to a room maintained at 76 ± 1 F and

50 ± 2 per cent relative humidity. At this time the vaseline was removed from the steel parts and measurements for the beginning of the drying period taken.

Testing the specimens: The testing was considered to have begun in most of the tests with the beginning of the drying period. The specimens may be divided into classes, according to the tests made, as follows:

Free shrinkage specimens were measured for length changes, weight losses and resonant frequency of vibration. Reference plugs were cast in the ends of the specimens for the length-change readings. They were hex-head cap screws arranged to give the desired gauge length. Ordinarily these were single plugs centrally located in the ends. But for those prisms that represented slabs drying from two opposite sides the arrangement was as shown in Fig. 31.

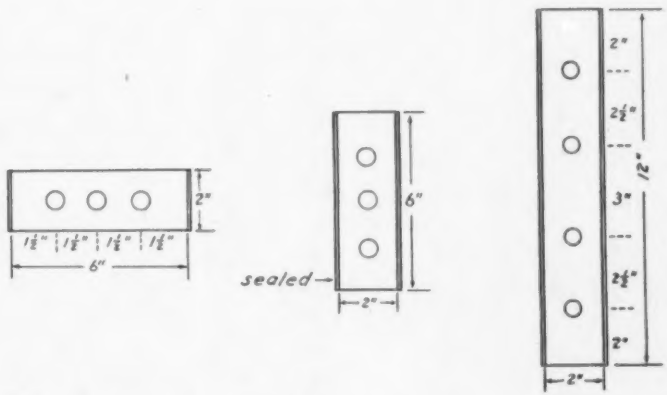


Fig. 31—End views showing arrangement of gage plugs in specimens representing 2-in., 6-in. and 12-in. slabs

The gage plugs for these specimens were ⅜ in. bolts 4 in. long.

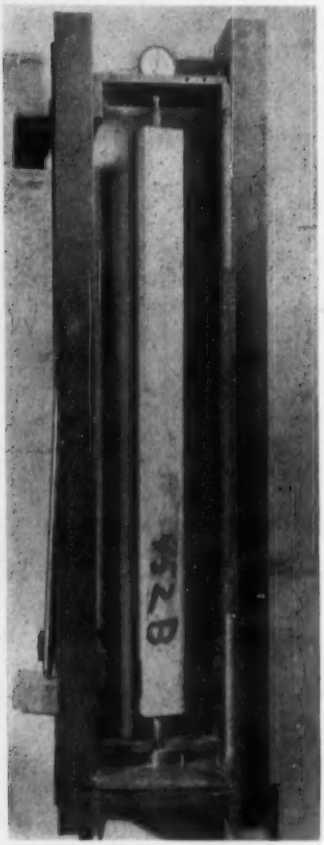
Restrained shrinkage specimens were partially restrained against shrinkage by steel restraining bars. The arrangement of the bars in those specimens that represented slabs drying from two opposite sides was the same as that shown in Fig. 31 for gage plugs except that only two bars 3 inches apart were cast in the 2x6-in. specimens. Square specimens that are permitted to dry from all four sides and that are partially restrained by one centrally located bar are used in routine testing of resistance to cracking. Further details in regard to restraining bars and the measurements for change in length of the restraining bar are shown in Fig. 32 and 33 as well as in Fig. 23 body of the text.

If the restrained-shrinkage specimen did not crack spontaneously before the maximum restraining force had been developed, additional increments of load were applied as shown in Fig. 33 until failure was produced.

Free warping specimens were measured for deflection over a 32-in. span. Most of these specimens were also measured for length change, weight loss and resonant fre-

quency. Some of the free warping specimens made in the early part of the work were not equipped for length-change measurements. Cross sections of free warping specimens are shown in Fig. 34. See Fig. 35.

Restrained warping specimens companion to the free warping specimens were loaded, as shown by Fig. 36, so as to prevent warping between the loaded quarter-points. Readings of the load required were taken periodically and, after the maximum restraint had developed, the load necessary to produce failure was determined.



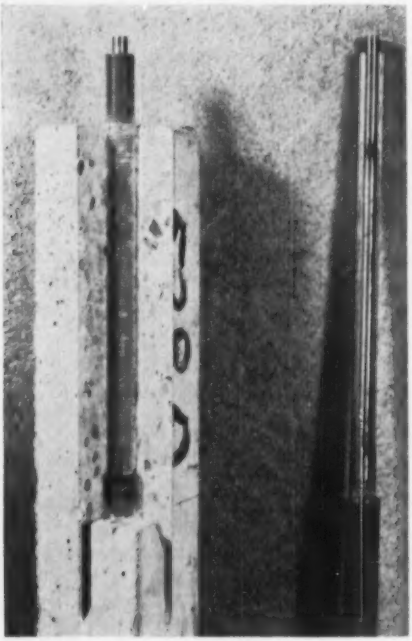


Fig. 32a (left)—Comparator
Fig. 32b (above)—Restraining Bar

Measurements of shrinkage-stress are made whether cracking occurs or not. The comparator (in Fig. 32a) is used to measure the changes in length of the steel restraining bar that result from the strains placed on it by the concrete. From these measurements and the known properties of steel, the average stress in the concrete is computed.

Important details of the restrained-shrinkage type of specimens are shown in Fig. 32b (See also Fig. 23 in the text.) Note that the concrete can grip the bar only in the end-region; contact in the 20-in. central section is prevented by a thick layer of rubber.

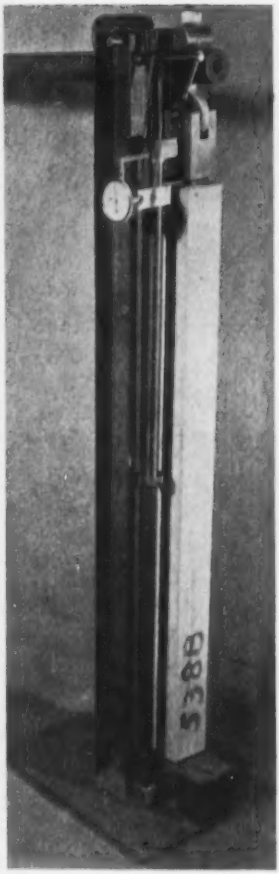


Fig. 33a

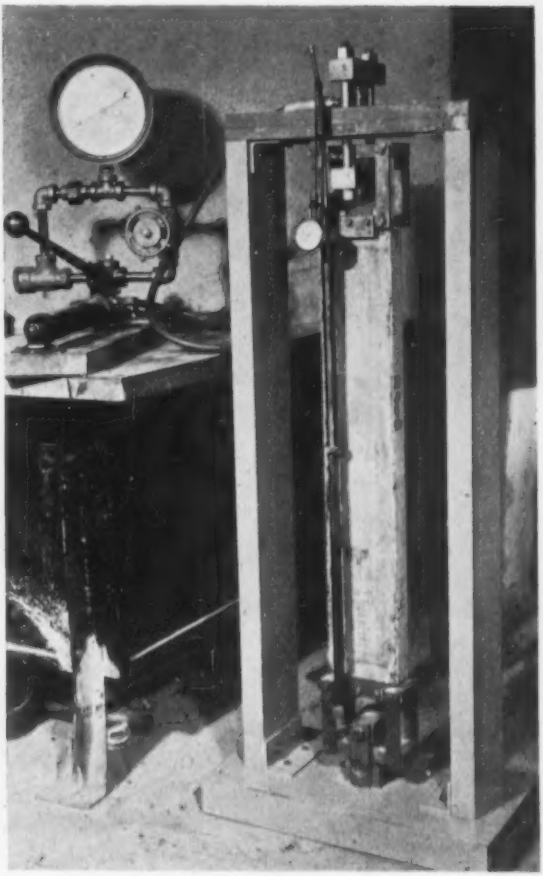


Fig. 33b

This method of test has been used to measure shrinkage-stress in specimens as wide as 12 in. and containing as many as four restraining bars.

Those specimens that do not fail under shrinkage-stress alone are given additional stress with the machines shown in Fig. 33a and 33b. The machine at the left (Fig. 33a) is used for most of the specimens; that at the right Fig. 33b is used if the capacity of the other one is exceeded and if the specimen contains more than one bar. The load is applied to the bar, and the extension of the bar at the time the concrete fails is determined by the strain-gage shown in the pictures. The net amount of load on the concrete at failure is computed from the magnitude of the load and the strain-gage reading. The factor of safety is the ratio of the load on the concrete at failure to the maximum load represented by the restraint against shrinkage.

"Sonic" testing: Most of the "unrestrained" specimens are tested periodically for frequency of vibration with the apparatus shown in Fig. 35a. Young's modulus is calculated from the resonant frequencies.

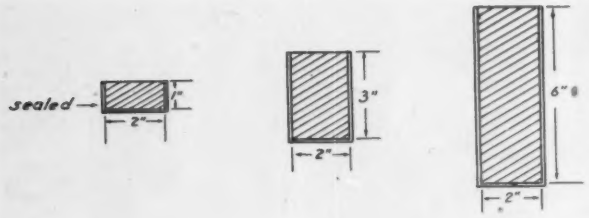


Fig. 34—Cross section of free warping specimens.

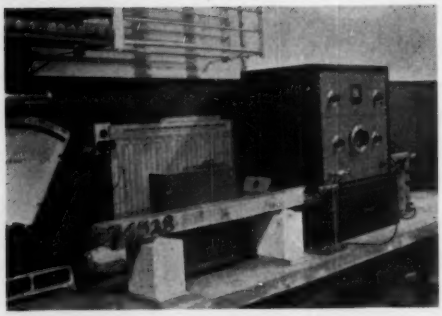
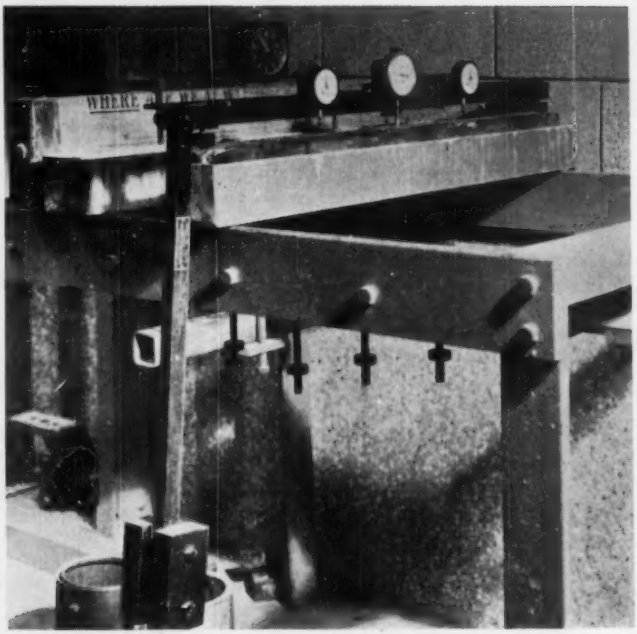
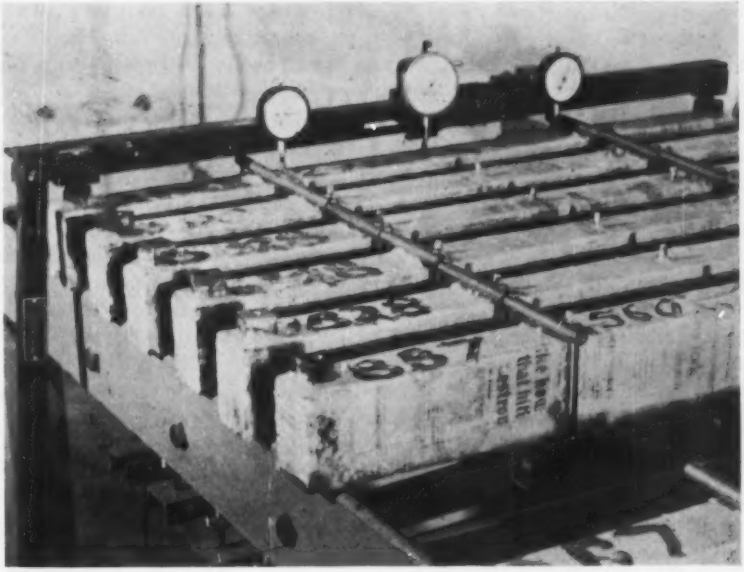


Fig. 35a—Sonic testing

Fig. 35b—Warping measurement





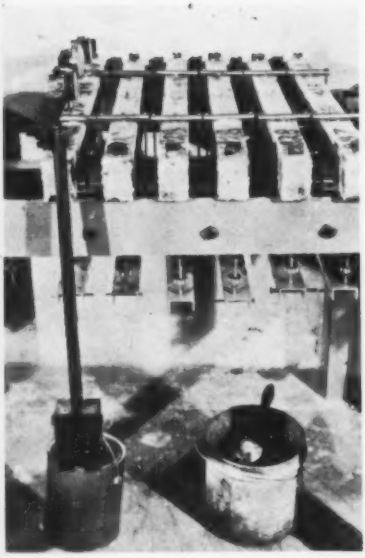


Fig. 36a (above)
Fig. 36b (left)
Restrained warping

Warping, length-change, and moisture-content: On specimens like that shown in Fig. 35a, the change in length is measured with the comparator shown in Fig. 32 and the warping is measured with the curvature-gage shown resting on a specimen in Fig. 35b. Results are correlated with concomitant changes in moisture-content.

Warping due to swelling: Previously dried specimens are placed, uncoated side down, in a trough of water as shown Fig. 35b or they are exposed to saturated air. The resulting warping due to absorption at one surface is more rapid than that due to shrinkage and can give rise to larger stresses.

The specimens shown Fig. 36 are supported only at the ends. They are coated on all but the bottom side and therefore as they dry they tend to bow upward. This tendency is opposed by the shackles at the quarter-points which are connected to a lever system below, one for each specimen. The levers are held by the fine-thread screw-adjustment seen best in Fig. 36b. The screws are turned downward until the force is just sufficient to prevent warping as indicated by the curvature-gage shown in both pictures. This instrument can be moved from specimen to specimen.

The force on the lever is measured periodically by finding the weight (bucket of shot) that will just hold the specimen in the position maintained by the screw. The force required reaches a maximum and then recedes slowly to zero as drying continues. When the maximum is reached, the specimens are loaded to failure. The ratio of the maximum force required to prevent warping to that required for failure is the factor of safety.

This test gives approximately the factor of safety against cracking of slabs of twice the depth of the specimens, drying equally from both sides.

A part of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE Vol. 42

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
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Vol. 17 No. 4 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN February 1946

Job Problems and Practice

Five cash awards—$50.00, $25.00 and 3 of $10.00 each are to be made for the best contributions to this department in the current volume year—Sept. 1945 to June 1946.

In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved —in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

Influence of Mixing Water "Hardness" on Air-Entrainment (42-172)

By CHARLES E. WUERPEL*

The question has been raised as to the influence of the "hardness" of the mixing water on the entrainment of air in a concrete mixture containing an air-entraining agent or admixture; it being believed possible that hard water in certain sections of the country might decrease the sudsing or air-entraining properties of neutralized Vinsol resin or similar agents with the result that less than the desired amount of air would be entrained in concrete with the normal amount of the agent.

The absence of an effect on air entrainment caused by the degree of hardness of the mixing water was demonstrated in a series of experiments in which the following waters, running the gamut of hardness, were used:

	Type of water	$CaCO_3$ p.p.m.
(a)	Distilled	0
(b)	Extreme softness in natural waters(1)	2.4
(c)	Mount Vernon, N.Y. tap water	20.0
(d)	Extreme hardness in natural waters(2)	1043.0

(1) Average lake water according to Data of Geochemistry by F. W. Clarke.
(2) Arkansas River. Data of Geochemistry, F. W. Clarke.

*Engineer in charge Central Concrete Laboratory, Corps of Engineers, Mt. Vernon, N. Y.

The waters (b) and (d) were prepared, in accordance with the method suggested and used by the Department of Water Supply, City of New York, by adding the equivalent amounts of calcium and magnesium in the form of $CaCl_2$ and $MgCl_2$ to distilled water.

Table 1 shows the results of tests made on mortar by A.S.T.M. Method C 185-44T using A.S.T.M. C 150-44 Type II cement with 0.010 per cent of neutralized Vinsol resin solution added to the water at the time of mixing.

Table 1

Water	Air Content—%
a	16.4
b	16.6
c	16.4
d	16.2

The lack of appreciable effect of the *Ca* and *Mg* ions present in the mixing water on the sudsing properties of the air-entraining agent is understandable when consideration is given to the much greater concentration of these ions which go into solution from the cement very rapidly after the water comes in contact with the cement. In other words, the hardness of the aqueous portion of a concrete mixture is so great as not to be appreciably affected by the hardness of the water supply.

Locating Points Along Beam Axis Corresponding to Known Moments (42-173)

By W. C. GOODWIN*

In the design of concrete building frames it is desirable to be able quickly to locate points corresponding to predetermined moment values. The location of points of inflection, points to bend or anchor reinforcement, or to change a concrete section are examples. The method submitted is not restricted to symmetrical† bending and applies to any combination of uniform and concentrated loads. It requires only that shears and moments be consistent with each other, the loading pattern, and conditions of restraint and continuity.

Notation

M	moment at a support.
M'	any part of M.
V	shear at support, consistent with M.
P	concentrated load at distance a from support.
x	distance to section X-X from support.
w	uniform load.

*Fabricated Steel Products Co., Boston.

†See Table V, "Continuity In Concrete Building Frames," Third Edition, Portland Cement Association.

Formulas

In Fig. 1, the moment at section X-X is

$M = Vx - P(x-a) - wx^2/2$ and

$$x = (M - Pa) / (V - P) + \{(wx^2 / 2(V - P)\} \quad (1)$$

If the load P does not occur within the distance x, then

$M = Vx - wx^2/2$ and

$$x = M/V + wx^2/2V \quad (2)$$

If neither the loads P nor w occur within the distance x, then

$M = Vx$ and

$$x = M/V \quad (3)$$

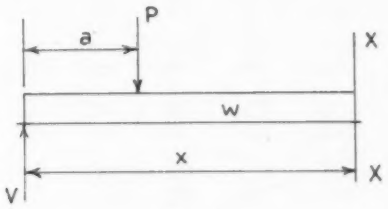


Fig. 1

Application

Case 1—When a beam in a building frame carries a concentrated load or loads near mid span and the weight of the beam is either negligible or for convenience included in the concentrations, formula (3) may be used directly for x.

Case 2—In the more common case where the beam carries a uniform load w either with or without concentrations, but where it may be seen by inspection that a is greater than x, formula (2) applies and x may be determined as follows:

Compute $q = M'/V$ and $r = 1000w/V$ separately. With their product rq enter Table 1 and select rx. Then $x = rx/r$.

Case 3—When a concentrated load occurs between the support and section X-X, formula (1) applies. Proceed as in Case 2 but substitute $M' - Pa$ for M' and $V - P$ for V.

$q = (M' - Pa) / (V - P)$ and $r = 1000w / (V - P)$.

In Table 1, values of $M/V = rq$ were computed for a constant $r = 1$ and various values of rx in the formula,

$x = M/V + wx^2/2V = q + rx^2/2000$,

When $r = 1$, $rx = rq + x^2/2000$,

For any value of r, $x = rx/r$.

Example 1—A beam with a span of 20 feet is loaded near mid span with a concentration of 100 kips which is assumed to include the weight of the beam. At one support the maximum negative moment is 300 foot kips and the shear is 52 kips. Find the point of inflection.

$x = M/V = 300/52 = 5.8$ feet.

TABLE 1

Values of rq and rx for $r = 1$

rq	10	20	30	39	49	58	68	77	86	95
rx	10	20	30	40	50	60	70	80	90	100
rq	104	113	122	130	139	147	156	164	172	180
rx	110	120	130	140	150	160	170	180	190	200
rq	188	196	204	211	219	226	234	241	248	255
rx	210	220	230	240	250	260	270	280	290	300
rq	262	269	276	282	289	295	302	308	314	320
rx	310	320	330	340	350	360	370	380	390	400
rq	326	332	338	343	349	354	360	365	370	375
rx	410	420	430	440	450	460	470	480	490	500
rq	385	394	403	412	420	428	435	442	449	455
rx	520	540	560	580	600	620	640	660	680	700
rq	461	466	471	476	480	485	489	492	495	500
rx	720	740	760	780	800	825	850	875	900	1000

Example 2—A beam with a span of 20 feet is loaded with 50 kips at each third point in addition to a uniform load of 2 kips per foot. At one support the maximum negative moment is 300 foot kips and the shear is 75 kips. (a) Find the point of inflection. (b) Find the point corresponding to 3/5 of the maximum negative moment.

(*a*) $q = 300/75 = 4.0$ $r = 2000/75 = 27$ $rq = 108$

$rx = 115$

$x = 115/27 = 4.3$ feet.

(b) $q = 3/5$ of $4.0 = 2.4$ $r = 27$ $rq = 65$

$rx = 67$

$x = 67/27 = 2.5$ feet.

Example 3—A beam with a span of 20 feet is loaded with a concentration of 50 kips located 2 feet from one support, in addition to a uniform load of 2 kips per foot. The negative moment at this support corresponding to the maximum positive moment is 140 foot kips, and the shear is 68 kips. Find the point of inflection for maximum positive moment.

$q = (140 - 50 \times 2) / (68 - 50) = 2.2$ $r = 2000 / (68 - 50) = 111$

$rq = 245$

$rx = 287$ $x = 287/111 = 2.6$ feet.

Fifth Annual

TECHNICAL PROGRESS SECTION

FOR COMMERCIAL PRODUCERS

February 1946

CONTENTS

Foreword 408

Advertisers and Advertising Subject Matter 409-472

Stearns Manufacturing Co., Inc. 409
Vibration and tamp type block machines, mixers and skip loaders

Blaw-Knox Division of Blaw-Knox Company 410-11
Truck-mixer loading and bulk cement plants, road machinery, buckets, and steel forms

A. C. Horn Company 412
Waterproofing

Baldwin Locomotive Works 413
Testing equipment

Raymond Concrete Pile Company 414
Pile foundations

Concrete Masonry Products Company 415
Non-shrink metallic aggregate

Electric Tamper & Equipment Company 416-17
Concrete vibrators

The Jaeger Machine Company 418-19
Concrete paving equipment

Kalman Floor Company 420-1
Floor finishing methods

Viber Company 422-3
Concrete vibrators

Koehring Company 424
Tilting and non-tilting construction mixers

Atlas Steel Construction Company 425
Forms for concrete

Scientific Concrete Service Corporation 426
Mix controls and records

Roberts and Schaefer Company 427
Thin shell concrete roofs

Lone Star Cement Corporation 428-9
Cement performance data

Chain Belt Company of Milwaukee 430-1
Mixers, pavers, pumps

Flexible Road Joint Machine Company 432
Finishing and joint installing machines

United States Rubber Company 433
Form lining

Fuller Company 434
Unloading and conveying pulverized materials

Ransome Machinery Co. 435
Paving mixers

Besser Manufacturing Co. 436
Concrete products plant equipment, production

The Prepakt Concrete Co., and Intrusion-Prepakt, Inc. 437-440
Pressure filled concrete

Mall Tool Company 441
Concrete vibrators

Vacuum Concrete, Inc. 442-3
Forms and lifters with suction controlled concrete

Whiteman Manufacturing Company 444-5
Vibrating and finishing equipment

Inland Steel Company 446-7
Reinforcing bars

The C. S. Johnson Co., ... 448
Mixing plant equipment

Hunt Process Company 449
Curing compound

Dewey and Almy Chemical Company 450-1
Air-entraining and plasticising agents

Butler Bin Co. 452
Mixing plants, cement handling equipment

The Calcium Chloride Association 453
Calcium chloride

Heltzel Steel Form & Iron Co. 454-5
Pavement expansion joint beams

Richmond Screw Anchor Company, Inc. 456
Planned form work

The Master Builders Company 457-464
Air entrainment and cement dispersion

American Concrete Institute 465
Publications about concrete

Master Vibrator Company 466-7
Concrete vibrators

Anti-Hydro Waterproofing Company 468-9
Waterproofing

Sika Chemical Corporation 470-1
Waterproofings and densifier

Alphabetical List of Advertisers 472

FOREWORD

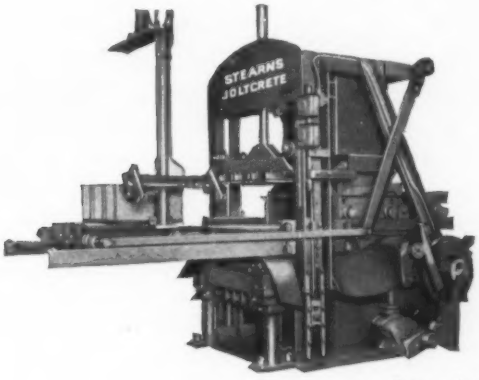
This is the Fifth Annual Technical Progress Issue of the ACI Journal and the first of the postwar period. It carries gratifyingly more advertising than any of its four predecessors.

To restate the original purpose and factors involved: Once a year in this section of the JOURNAL the seller of goods and services in the field of concrete would reach the potential buyer with an opportunity for mutual benefit. The technically trained engineer-buyer is scornful of ballyhoo, alienated by overstatement, breezy generalities or by any highly imaginative sales promotion. The man responsible for sales is likely to be close to the advertising department. To what degree can commercial advertising "copy" be made to read like engineering good sense? Could the salesman's claims be backed up with facts?

The Publications Committee has stood guard. The surprising facts of the experience are two: 1) The small percentage of "copy" submitted which has been sent back to its source for revision, and 2) the willingness of advertisers, almost without exception, to meet the Publications Committee's suggestions to reduce the "claims" for a product to statements which could be, at least partially, authenticated. Advertisers have seemed to see that claims for their products which put a severe strain on credulity of the intelligent reader *just aren't good advertising.*

This is not to say that the Publications Committee critics are hard-boiled—by no means. They are able to see that many claims which are a long way from *under*-statement still do not *deceive* anyone. These critics know, and know that the ACI public knows, that sales viewpoint *may* be arrived at through over-rosy spectacles. Can the copywriter be expected to put out a message that is as completely qualified as the typical researcher's well hedged conclusions!

Sometime there may be an accepted buyer-seller esperanto, or a basic English of the marts of trade. Until then buyers will have some vestigial reminder of the classic warning, "Let the buyer beware!" Seriously, however, while ACI endorsement of the pages which follow is neither given nor implied, most of our advertising seems to be written against a background of performance and with the knowledge that, in the long view, there are profits only where there is authentic "straight talk".



Stearns Improved
Joltcrete No. 9

THE EXCELLENCE OF PRODUCTION that won four Army-Navy "E" awards for Stearns is again at work in its own, its original field: the design and manufacture of concrete products plant equipment.

• STEARNS MACHINES, vibration and tamp-type block makers, mixers and material handling skip loaders will solve any post-war production problem that concrete products manufacturers may have.

• STEARNS ENGINEERS AND DEALERS will continue to give this vital industry the benefit of their mature experience.

• MAKE USE OF BOTH!

ARMY E NAVY

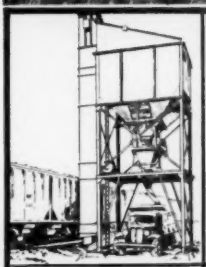
STEARNS

Pioneers in the development of new and improved equipment for the volume production of concrete masonry units.

BLAW-KNOX DIVISION of Blaw-Knox Company
Farmers Bank Bldg. • Pittsburgh, Pa.—New York, Chicago, Birmingham, Philadelphia, Washington

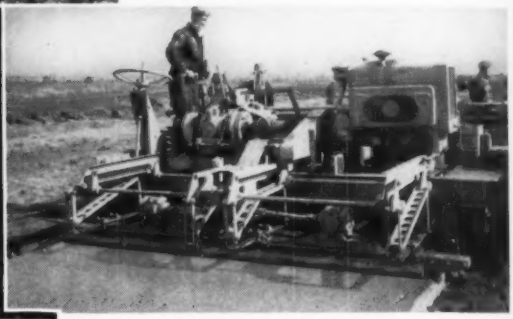
USE THIS UP-TO-DATE

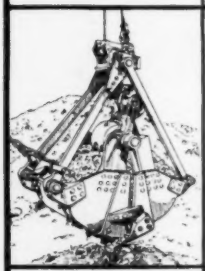
Dry, harsh, concrete paving mix being handled by Blaw-Knox Transverse-Blade Automatic Type Concrete Paving Spreader equipped with vibratory attachment. Concrete tested ½ to ¾ inch slump. Contractor's production in spite of difficult concrete was in excess of 400 lineal ft. of 12 ft. wide slab 9" thick per hour. Spreader-Vibrator is one man operated. Vibration increased strength of concrete by 25 per cent.



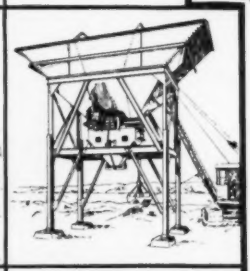
Bulk Cement Plants

Concrete Buckets



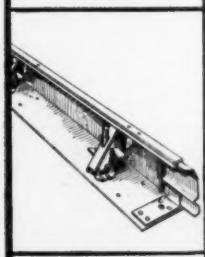


Clamshell Buckets

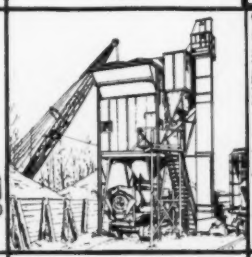


Aggregate Batching Plants

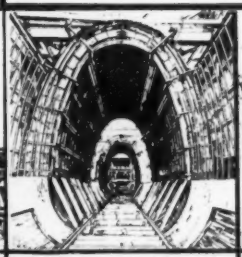
View behind Blaw-Knox Spreader-Vibrator shown in upper photograph. Concrete has been spread to required elevation and simultaneously compacted by vibratory attachment. Note uniformly smooth surface behind vibrator. Blaw-Knox Finishing Machine worked closely behind Spreader-Vibrator and kept pace easily. Cores drilled from completed pavement showed no honeycomb at bottom of slab or at joints and no excess mortar at surface of pavement.



Paving Forms for Roads and Airports



Truck Mixer Loading Plants



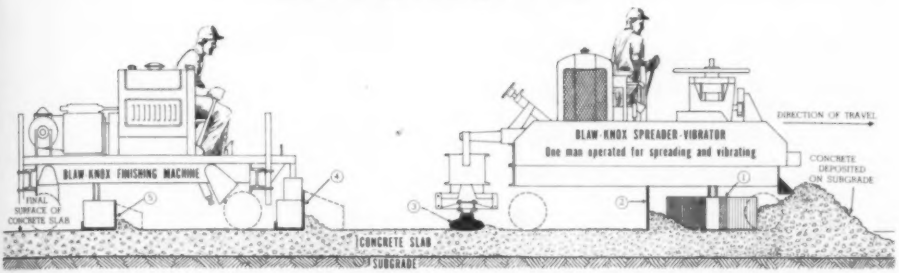
Steel Forms for General Concrete Construction

Steel Street Forms

• BLAW-KNOX •

PAVING METHOD

for greater production . . . lower cost and higher quality



1 Automatic Transverse Spreading Blade spreads concrete transversely and at the same time pushes excess concrete ahead of machine; adjustable for spreading height.

2 Strike-off shapes concrete to required height and crown allowing slight excess for compaction by vibrator; strike-off is hydraulically adjustable for elevation.

3 Vibratory attachment compacts concrete simultaneously with spreading operation; vibrator is spring suspended and does not rest on side forms. All vibratory effect is transmitted directly to the concrete. Vibrator is controlled by spreader operator and leaves slight excess of concrete for finishing machine.

4 Finishing Machine front screed strikes off excess of concrete to exact grade and crown. Finisher has easy and rapid operation; follows close behind Spreader-Vibrator.

5 Rear screed of Finishing Machine performs final finishing and smoothing operation.

The method of paving construction illustrated has been proved on hundreds of miles of concrete paving construction for roads and airports.

The dry and harsh concrete mixes frequently specified by engineers for modern pavements can be spread, compacted and surfaced most rapidly and efficiently by the combination of the Blaw-Knox Transverse-Blade Type Automatic Concrete Paving Spreader equipped with vibratory attachment and the modern Blaw-Knox Finishing Machine.

The Spreader-Vibrator spreads the concrete to the required depth and at the same time compacts the concrete by vibration. The Finishing Machine follows close on the heels of the Spreader-Vibrator and does a quick and easy surfacing job. The Blaw-Knox Spreader-Vibrator teamed with the Blaw-Knox Finishing Machine handles the output of two 34-E dual drum paving mixers.

Difficult concrete is easily handled on a production basis by this up-to-date paving method and the contractor gains — in greater yardage, lower construction cost, minimum of manual operations and higher quality paving.

The Blaw-Knox Finishing Machine can also be equipped with a vibratory attachment. However, experience has shown that the paving vibrator mounted on the spreader provides better compaction, more practical operating procedure, and maximum production of paving slab. The Spreader-Vibrator always remains with the paving mixer and does not have to move back to aid in correction of high or low areas.

Blaw-Knox Spreaders and Finishers including vibratory attachments are available in standard sizes as follows: 10-15 ft. adjustable width, 20-25 ft. adjustable width.

Your Nearest Blaw-Knox Distributor Will Promptly and Efficiently Handle Your Inquiries for Construction Equipment.

RAINCOATS FOR BUILDING EXTERIORS

Ten years ago our scientists started on the problem of finding a more effective way of restoring the masonry surfaces of concrete, stucco or brick buildings. Today America's buildings are able to profit from our long research. For we have created a coating which bonds itself mechanically and chemically to exterior surfaces, yet lets the masonry breathe. It is called Waterfoil. There is no other protective coating like it.

Waterfoil is manufactured of irreversible inorganic gels . . .which forms a hard, heavy, fine textured coating. Water vapor may escape, but water absorption is impeded, thus preventing reinforcing bar rust and spalling. With Waterfoil you can decorate and restore your disintegrating masonry walls . . . and preserve your buildings for tomorrow. Any careful workman can apply Waterfoil. Send for the literature on Waterfoil . . . it's important.

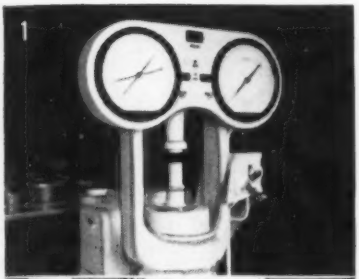
A. C. HORN COMPANY

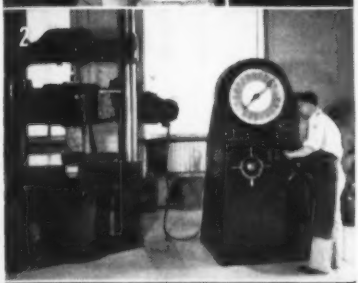
Established 1897

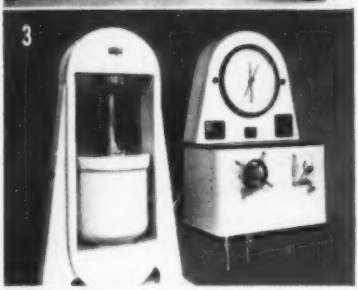
Manufacturers of Materials for Building Maintenance and Construction

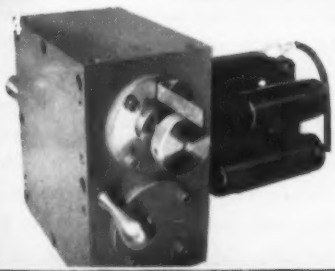
LONG ISLAND CITY 1, N. Y.

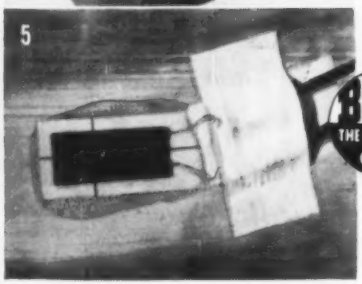
HOUSTON, TEXAS ● CHICAGO, ILLINOIS ● SAN FRANCISCO, CALIF.











TESTING EQUIPMENT FOR CONCRETE CONSTRUCTION

Today's construction projects classify their physical testing equipment three ways. One, testing materials of construction in plant laboratories; two, testing as a means of concrete quality control in field laboratories; three, testing in place.

The Baldwin Locomotive Works offers equipment for all three purposes: For materials of construction the 90,000# compression machine of Figure 1 is especially built for 2" x 2" cubes, small cylinders and beams and will be found in use wherever compression testing of cement and concrete cubes and cylinders is practiced.

Figure 2 is a typical 400,000# universal testing machine installation for plant laboratory service, fitted with the Emery capsule and the Tate-Emery null method multi-range dial indicator. This machine is designed for testing concrete in compression and reinforcing steel in tension.

For field laboratories the 400,000# compression machine in Figure 3 is small, accurate and convenient to handle. This is a single purpose machine of all welded plate construction with packless ram and Emery null method dial indicator which makes four revolutions of 100,000# per revolution.

For dynamic testing of structures in place we introduce the Lazan Mechanical Oscillator of Figure 4, useful not only for vibratory testing of a completed structure to simulate service but for consolidation purposes and any other use where high frequency mechanical vibrations will serve.

For experimental stress analysis in such structures, SR-4 strain gages, Figure 5, are pre-eminent and can be used in many ways. Baldwin is ready to furnish details on any of the above. The Baldwin Locomotive Works, Philadelphia 42, Pa., U. S. A. Offices: Philadelphia, New York, Chicago, Cleveland, Washington, Boston, Detroit, St. Louis, Houston, Pittsburgh.

BLW THE BALDWIN GROUP

BALDWIN

SOUTHWARK

TESTING EQUIPMENT

IT'S THE "Know-how" THAT COUNTS

● When you call in the Raymond organization, you are turning your foundation contract over to men who are expertly trained and thoroughly experienced in every scope of the work. They know their jobs—from the preliminary investigation of underground conditions to the complete installation of the foundation.

They also know their equipment—how to handle the largest rig and the smallest tool needed in the efficient placing of Raymond Piles. And, too—they know their responsibility for carrying out large and small jobs to successful completion.

Since 1897 Raymond Concrete Piles have been continuously and increasingly used by the architectural and engineering professions. Today over 50 million lineal feet of these sturdy cast-in-place piles are in world-wide use. With over 11,000 contracts to its credit, Raymond has accumulated a vast fund of knowledge, experience and ability. Write, wire, cable or phone for a competent Raymond engineer to discuss your next project with you.

SCOPE OF RAYMOND'S ACTIVITIES

includes every recognized type of pile foundation—concrete, composite, precast, steel, pipe and wood. Also caissons, construction involving shore protection, ship building facilities, harbor and river improvements, borings for soil investigation.

RAYMOND CONCRETE PILE CO.

140 CEDAR STREET • NEW YORK 6, N. Y.

Reg. U. S. Pat. Off.

Non-Shrink Metallic Aggregate

KEMOX is an outstanding product specially formulated to perform difficult tasks in the field of surface and integral waterproofing, grouting and general Industrial Maintenance.

Some of the components of Kemox went to war, consequently it has been out of production, but will soon be available in improved form. **KEMOX** has been successfully employed in Government projects, and by Municipalities and Industry. Testimonials attest to Kemox as a dependable medium by which to solve difficult and troublesome industrial maintenance problems—

Inquiries invited.

CONCRETE MASONRY PRODUCTS COMPANY

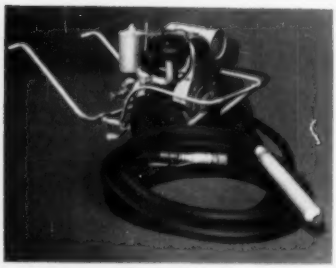
140 West 65th St. - - Chicago

JACKSON—The RIGHT

For each and every purpose to which Vibrators are applicable in the concrete industry, we are confident we can supply the equipment that will give you not only the best and fastest placement, but also the maximum of dependability and trouble-free service. Pioneers and outstanding developers of vibratory equipment, our complete line includes internal and external vibrators for: General Construction ★ Light Construction ★ Mass Concrete Dam Construction ★ Hard-to-get-at Places ★ Form Vibrating ★ Floors, Streets and Highways ★ Pipe Manufacturing ★ Movement of Materials—Vibratory Tables, etc. Drop us a line for the best solution to any concrete vibrating problem.

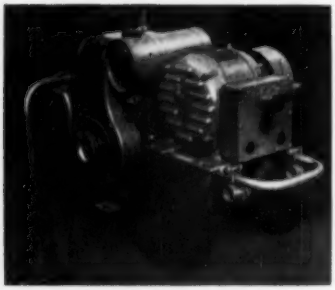
JACKSON HEAVY-DUTY VIBRATORY PAVING TUBE

For speedy full width highway and airport concrete paving. Up to 25 feet widths. Submergible dual tubes energized by powerful vibratory motors quickly transform harsh mixes to plastic state. Assures complete compaction—easy finishing. Cement savings up to 10% through reduction in W/C ratio. Attaches to any modern finisher. Variable frequency 3000-5600 V.P.M. Hydraulic lift. Grouped controls.



HS-A1 HYDRAULIC CONCRETE VIBRATOR

This is a general purpose machine of the internal type adapted to a wide range of applications. Operated by light oil pumped through hose line 34 feet long to hydraulic motor in vibrator head. Valve on power plant adjusts frequency desired, from idling to top speed of 7200 V.P.M. All moving parts in the hydraulic medium (oil). Vibrator head 2¾" diameter, standard, gas engine, air cooled 4.7 H.P. A general favorite because of its wide range of application and low maintenance.



JACKSON PORTABLE POWER PLANTS

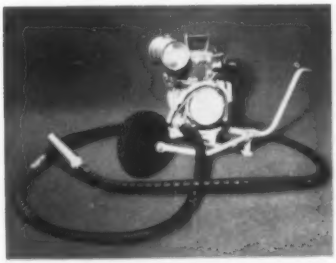
A new high in dependable portable power is now available in Jackson postwar Power Plants having permanent magnet generators in which all usual maintenance is eliminated except lubrication. Especially designed for severest service under continuous operating conditions. Simple, rugged construction and design, stripped of fussy control gadgets. Run all Jackson Vibrators, for lights, and all types of contractor's power tools. One to 7.5 KVA sizes available.

ELECTRIC TAMPER & EQUIPMENT CO.,

Answer

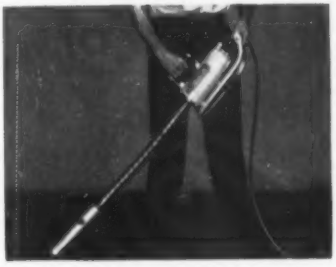
to EVERY VIBRATOR PROBLEM in the Concrete Construction Field

FS-6A GAS-DRIVEN, FLEXIBLE SHAFT CONCRETE VIBRATOR



For placing concrete in wall, column and slab sections of ordinary size. The smaller vibratory head is exceptionally well adapted to concrete placement in thinner sections. 3 H.P. air-cooled gas engine. Automatic clutch, V-belt drive. Countershaft has oilite bearings. Turntable base, dirtproof. Optional wheelbarrow with drop down lifting handles. Flexible shaft drive. Vibrator head $2\frac{3}{8}$" diameter x $18\frac{1}{2}$" long or $1\frac{13}{16}$" x 17". Frequency 7000 to 7200 V.P.M. variable with length of shaft, size of head, and concrete consistency.

FS-7A ELECTRIC-DRIVEN, FLEXIBLE SHAFT CONCRETE VIBRATOR



A truly general construction vibrator which because of its exceptionally powerful but lightweight motor, will operate any of our standard vibrator heads ($2\frac{3}{8}$" x $18\frac{5}{8}$"; $1\frac{13}{16}$" x $16\frac{7}{8}$"; $1\frac{13}{16}$" x $16\frac{7}{8}$"; $1\frac{13}{16}$" x $10\frac{3}{4}$") with shaft lengths of 24", 36", 7', 14' and 21'. Universal motor, operates on 115 Volt A.C. or D.C. and will deliver 7000 to 10,000 V.P.M. depending on the consistency of concrete, head and length of shaft employed. Does the work of many larger vibrators.

ONE MAN OPERATED ELECTRIC VIBRATORY HAND SCREED

Practically self propelled in forward direction. Second passes made by simply rolling back necessary distance. Gets right up to walls. Variable frequency control, to suit consistency of any mix and second passes, may be had by instant adjustment of power plant engine throttle (low as necessary to 85 cycles). Assures thorough puddling throughout section. Most convenient, time and money saving vibratory hand screed on the market.

LUDINGTON, MICHIGAN

Anticipating Tomorrow's

1 CONCRETE RE-MIXED ON THE SUB-GRADE by compacting spreader screw:

Comparative tests by highway engineers of various States have proved conclusively that the Jaeger method of screw-spreading concrete produces a more uniform, denser and, therefore, longer wearing slab.

By its thoro and positive re-mixing and inter-mixing of piles dumped on the subgrade, both the segregation of coarse aggregates in the batch and the variations between different paver batches are eliminated; badly placed batches are redistributed to leave a uniform spread of material ahead of the Finishing Machine, with material placed so solidly against the road base and side forms as to eliminate the honeycomb problem and the entire mass compacted to weight and density approaching that of vibrated concrete.

As one prominent engineer states: "It has been demonstrated that the quality of concrete can be improved and at the same time cost of production to the contractor can be reduced."

It seems logical to expect that re-mixing on the subgrade will be specified where it is desired to insure highest strength, longest life pavements.

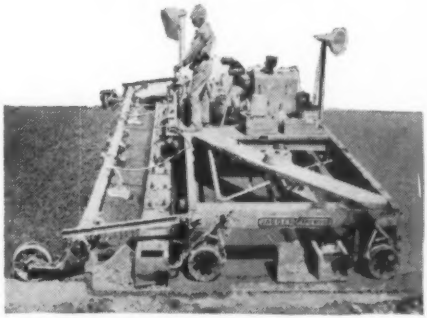
THE JAEGER MACHINE COMPANY, COLUMBUS, OHIO

Paving Specifications . . .

2 VIBRATION ON THE FINISHER, not on the concrete spreader:

Although Jaeger can furnish a vibratory attachment for use on Spreaders, the recommended Jaeger method of vibration *on the Finisher* has proved superior for any true vibratory mix. On an efficiently run job, only the Finisher has time to go back for more than one vibratory pass, as often needed. Also, it is the machine which always finishes to form level, thus insuring an over-all vibrated surface.

It has been demonstrated that specifications for true vibratory mixtures can best be met by use of the Jaeger Vibratory Finisher with "bullnose" screed giving DEEP INTERNAL VIBRATION. For less difficult mixtures use of a vibratory tube, *on the Finisher,* is also satisfactory.

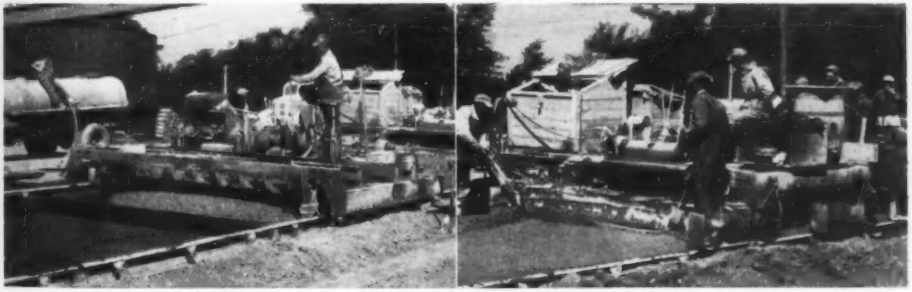


3 FAST, MECHANIZED PLACEMENT and FINISHING of quick-drying air-entraining cements:

The Spreader-Finisher "team", originated by Jaeger, which made it possible to handle stiff, vibratory concrete at the dual-drum paver pace, also equips road builders to handle quick-drying air-entraining cements.

Under hot, windy or dry air conditions, the Jaeger "team" provides the spreading and finishing capacity needed to keep close behind big pavers and complete the job before drying hinders a satisfactory finish.* Also, an exclusive feature of the Jaeger Finisher is that screed speeds can be independent of traction, permitting the use of fast screed speeds which has been found necessary to prevent tearing when the surface is sticky.†

* † Discussions of these two problems have appeared in previous issues of the Journal of the American Concrete Institute.



Manufacturers of Concrete and Bituminous Spreading and Finishing Equipment, Road Forms, Concrete and Bituminous Mixers, Truck Mixers, Pumps, Air Compressors, Hoists, Material Loaders.

KALMAN

ABSORPTION-PROCESS

Cross Section of Kalman Floor Topping,
Showing Uniform Distribution of Aggregate and Density

THE KALMAN FLOOR—A GRANOLITHIC CEMENT FINISH FLOOR TOPPING. Laid by the Kalman Floor Company using the KALMAN ABSORPTION PROCESS.

The Kalman Floor Company has for 28 years pioneered and specialized in the development and installation of Granolithic Cement Finish Floor Topping and has during this period maintained continuous leadership in the field through scientific research and experiments.

KALMAN FLOOR COMPANY
INCORPORATED
110 EAST 42ND ST. NEW YORK 17, N. Y.

FLOOR

CEMENT-FINISH

The Kalman Absorption Process

A practical method of installing on a commercial basis a theoretically correct low-water concrete topping. This method, combined with the skill and knowledge of the men doing the work and subjecting all materials to Laboratory test for analysis, gradation and soundness, plus a constantly growing skilled personnel and improvement in equipment has consistently produced a uniformly hard wear-resisting floor, free from disintegration and dusting and of a maximum density, evenness of texture and unexcelled durability.

Adaptability

Over 200,000,000 Sq. Ft. of Kalman Floors have been installed throughout the United States in practically every type of Industrial, Warehousing, Institutional, Commercial and School buildings and is readily adaptable. (For further information, see Sweet's Architectural Catalog. A. I. A. File No. 4 i 3.)

OFFICES:

NEW YORK	CHICAGO	BOSTON
LOS ANGELES	PHILADELPHIA	CHARLOTTE
SEATTLE	CLEVELAND	DAYTON

UNMATCHED ADVANTAGES... of VIBER Vibrators

9,500 RPMs in concrete. This is the speed of Viber Vibrators—the highest ever offered. Unparalleled economies in quality concrete construction are provided by the compacting efficiency of this internal high speed, *full depth* vibration, and better workmanship is the result.

1 **Faster** construction, because Viber efficiency permits the use of drier mixes and larger sized aggregates—and saves time in many ways.

2 **Positive** bonding and consolidation . . . from smooth, steady vibration free from the destructive slugging action that displaces material and forms.

3 **Uniform** density and strength . . . a result of strong, constant vibration that is effective in areas sufficiently large to assure overall consolidation.

4 **Complete** Compaction . . . in less time, at less cost.

5 **Increased Labor Efficiency** results from the ease with which Viber Vibrators may be handled without strain, for one man operation. Workmen like to use Vibers and are less likely to do careless work.

6 **Freedom from breakdowns.** Viber equipment is constructed for heavy duty, it is very sturdy and remarkably free from mechanical troubles.

7 The **INTERCHANGEABILITY** of all Viber units reduces the investment in equipment, saves time in changing to different jobs. Viber units are powered by electric, pneumatic or gasoline motors.

KEEP AHEAD OF SCHEDULE WITH VIBERS

VIBER COMPANY

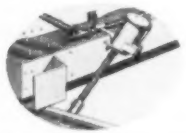
726 SOUTH FLOWER STREET • BURBANK, CALIFORNIA

ORIGINATORS OF INTERNAL CONCRETE VIBRATION!

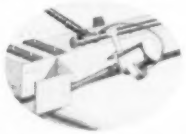
The battery of *high-speed* VIBER VIBRATORS is one-man operated. A single hydraulic control valve raises or lowers the vibrators as a single unit, for efficient operation in desired positions.

FASTER CONSTRUCTION OF SUPER HIGHWAYS AND RUNWAYS

(A) Viber Vibrators extended in upward position, giving ample clearance to pass over joint, manhole covers or other obstacles.



(B) Position of Vibrators starting under oscillating screed.



(C) Position of Vibrators submerged under oscillating screed to full depth of concrete slab.

AS FAST AS IT CAN BE PLACED, even the stiffest concrete of super-thick slabs can now be economically vibrated *full depth*, for uniform density and strength. Higher quality concrete, with substantial savings in time, labor, construction and maintenance costs result from use of the VIBER SLAB. This multiple unit, high-speed internal vibrating machine is specially designed for *continuous full width*, full depth compaction of slabs up to 25 feet. The VIBER SLAB requires but a single operator and may be attached to most types and makes of spreading and finishing equipment. A battery of correctly spaced VIBER VIBRATORS give you the unequalled speed, *in concrete*, of 9,500 RPMs, and test borings prove their unmatched compacting efficiency. Full details will be furnished upon request.

VIBER SLAB

MULTIPLE HIGH-SPEED INTERNAL VIBRATION

VIBER COMPANY • 726 South Flower St. • Burbank, California

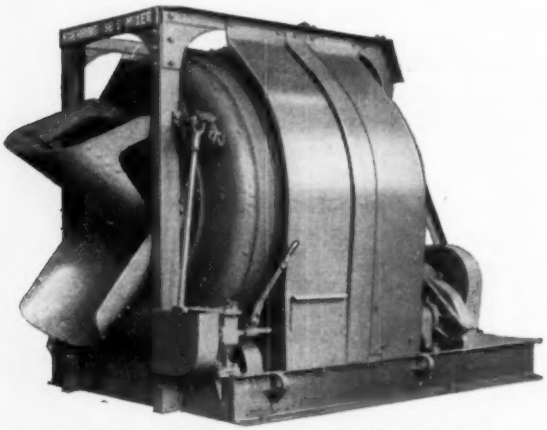
ORIGINATORS OF INTERNAL CONCRETE VIBRATION!

KOEHRING

TILTING AND NON-TILTING HEAVY-DUTY CONSTRUCTION MIXERS

Koehring Concentric Zone Tilting Mixer
56-S, 84-S, 112-S

Important progress in the design of Koehring Tilting Construction Mixers during the past years has been made to further the production of better concrete . . . concrete best suited for the large volume dam structures and other similar projects. These improvements, because they are responsible for better concrete, also permit concrete production in a shorter mixing period. Large drum single opening, 15° above horizontal mixing position and 60° below horizontal discharge position are a few of the important progress changes in the design of Koehring Tilting Mixers.



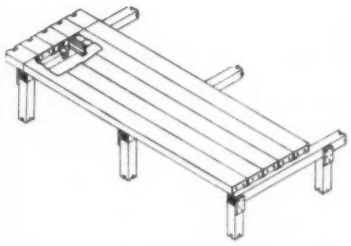
Koehring Non-Tilting Mixer 28-S, 56-S, 84-S

Tilting mixers are built in sizes 2, 3 and 4 cubic yards of mixed concrete and can be used in concrete plants singly or in batteries of 2, 3, 4 or 5 using the concentric zone plan for discharge into one central hopper. Charging is also from one central group of bin and batchers.

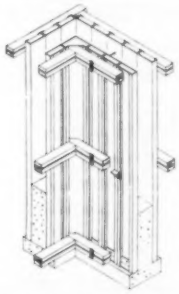
Non-Tilting Mixers are available in sizes of 1, 2 or 3 cubic yards per batch. The Koehring Flow-Line discharge chute for minimum of segregation is one of many recent design improvements.

KOEHRING COMPANY - - Milwaukee, Wis.

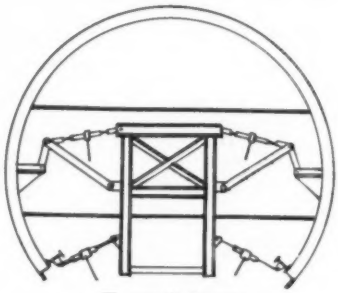
Labor-Saving Concrete Forms



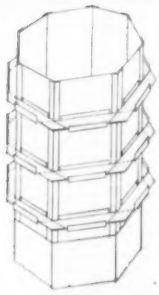
Flat Slab Forms



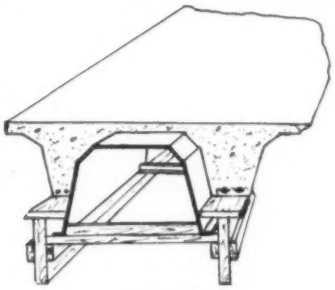
Travelling Wall Forms



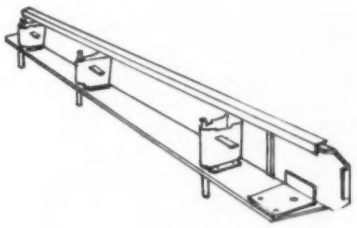
Tunnel Forms



Column Forms



Pan Forms



Road Forms

Also, Centering, Pipe Forms, Slide Forms, Shaft Forms, Manhole Forms, Etc.

Plants: Irvington, N. Y. Lancaster, Pa. Zanesville, Ohio

ATLAS STEEL CONSTRUCTION COMPANY

83 James Street - - - IRVINGTON, N. Y.

Atlas Steel Forms for every Purpose

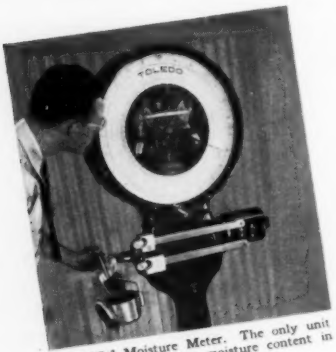
FORMULA FOR CONCRETE

Mix Design + Batching + Transportation + Placing and Curing = ?

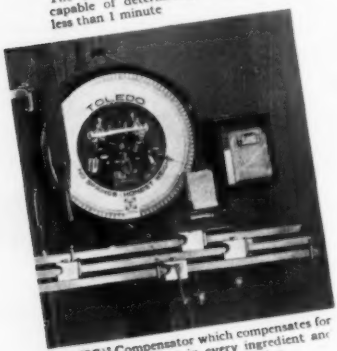
Engineers know how to analyze materials and make scientific mix-designs that will produce any desired qualities when mixed in the concrete laboratory

Modern transportation and placing equipment are available. The *weak link* is batching. Many plants still use equipment of the type first used when the change from volumetric to weight batching was made and concrete batched with such equipment frequently bears little or no resemblance to the carefully prepared mix-design.

Modern scientific batching equipment to fill this gap is available. Specify [SC]² PRECISION CONCRETE CONTROL and batching accuracy is assured. It includes:



The (SC)² Moisture Meter. The only unit capable of determining moisture content in less than 1 minute



The (SC)² Compensator which compensates for the moisture content in every ingredient and graphically records its every operation

MOISTURE DETERMINATIONS, accurate to ¼% made in one minute; one for every batch when necessary.

AUTOMATIC COMPENSATION FOR MOISTURE CONTENT in fine and coarse aggregates. Does not require calculations, charts or tables. With it, any intelligent weigh-man can produce remarkable uniformity.

PRECISION PROPORTIONING OF ALL INGREDIENTS using TOLEDO, the world's most accurate scales. Automatic cut-off is used for cement and water and is optional for aggregates.

AUTOMATIC GRAPHIC OR PRINTED RECORDS OF EVERY BATCH showing the delivered weight of each ingredient.

[SC]² CONTROL is approved by every concrete engineer, ready-mix operator and contractor who has ever used it.

Write today for information on how engineers are specifying and getting accurately controlled concrete.

Scientific Concrete Service Corporation
724 Salem Ave. **Elizabeth 3, N. J.**

SCIENTIFIC [SC]² CONCRETE

TRADE MARK REG. — SCIENTIFIC CONCRETE SERVICE CORPORATION WASHINGTON D.C. — U.S. PATENT OFFICE

THIN SHELL CONCRETE ROOFS

SPEEDY, ECONOMICAL, ADAPTABLE, TIME TESTED, FIRE PROOF

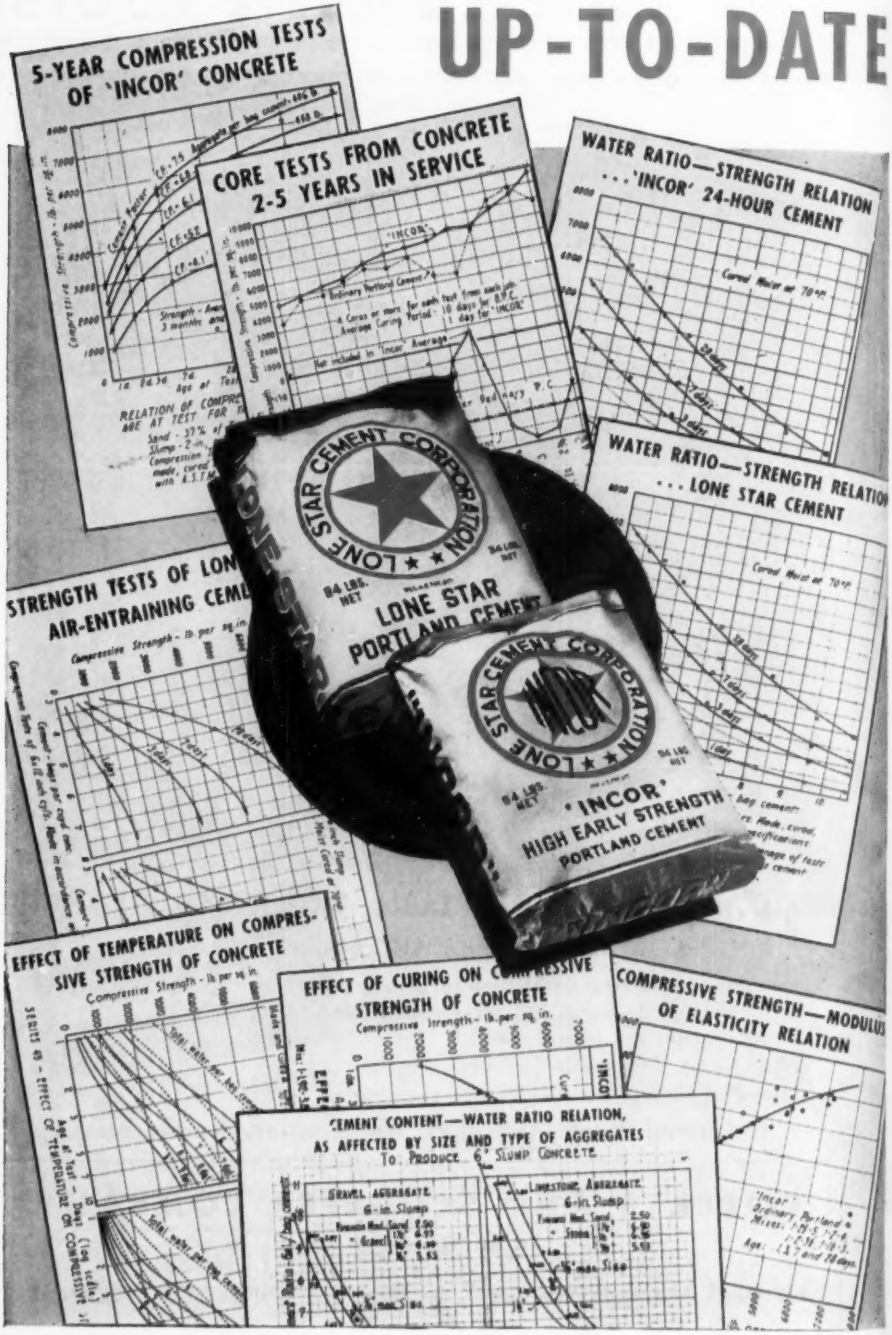
LOW MAINTENANCE AND INSURANCE RATES

Concrete forms roll on own wheels for greater reuse and reduction in cost.
Smooth underside of roof and unobstructed useful space.
Fewer columns and greater spans.
Beam action of curved roof utilized.
Local materials and local labor used.

Industrial Plants, Garages, Warehouses, Auditoriums

ROBERTS AND SCHAEFER COMPANY
ENGINEERS

307 North Michigan Avenue **CHICAGO 1, ILLINOIS**


UP-TO-DATE
5-YEAR COMPRESSION TESTS OF 'INCOR' CONCRETE
CORE TESTS FROM CONCRETE 2-5 YEARS IN SERVICE
WATER RATIO—STRENGTH RELATION . . . 'INCOR' 24-HOUR CEMENT
WATER RATIO—STRENGTH RELATION . . . LONE STAR CEMENT
STRENGTH TESTS OF LON AIR-ENTRAINING CEM
EFFECT OF TEMPERATURE ON COMPRESSIVE STRENGTH OF CONCRETE
EFFECT OF CURING ON COMPRESSIVE STRENGTH OF CONCRETE
COMPRESSIVE STRENGTH—MODULUS OF ELASTICITY RELATION
CEMENT CONTENT—WATER RATIO RELATION, AS AFFECTED BY SIZE AND TYPE OF AGGREGATES To Produce 6" Slump Concrete
LONE STAR CEMENT CORPORATION
94 LBS. NET
LONE STAR PORTLAND CEMENT
'INCOR' HIGH EARLY STRENGTH PORTLAND CEMENT

PERFORMANCE DATA

CONTINUOUS research and rigid manufacturing control assure utmost quality and uniformity of Lone Star Cements. Lone Star Cement Research Laboratories furnish up-to-date performance data to help the user obtain best results in concrete — durability as well as strength. Lone Star Cements cover the entire construction range:

Lone Star Cement for normal schedules, where time is available for curing and hardening.

'Incor' 24-Hour Cement, where reduced curing time and dependable high early strength provide important advantages.

Lone Star Air-Entraining Portland Cement for specific requirements in highway and other uses.

Select cement to fit the job: Use Lone Star Cement or 'Incor', whichever shows the lower cost of concrete in place and ready to use. Field experience since 1927 proves 'Incor'* advantages on—

STRUCTURES: 24-hour form removal, 50% less forms, no reshoring, ready for immediate use.

HEAVY-DUTY FLOORS: Overnight service, superior wear-resistance with only 24 hours curing. For severe exposures, cure 48 hours.

WATERTIGHT CONCRETE: Thorough curing in 1-2 days, saves 5-7 days. Smoother-working mixes place easier, no separation.

WINTER WORK: Now economical; only 1-2 days heat curing; saves forms, heating, tarpaulins.

Let job analysis decide — selective concreting assures quality concrete at less cost. Write nearest office for performance data for use in designing mixes and planning schedules.

*Reg. U. S. Pat. Off.

LONE STAR CEMENT CORPORATION

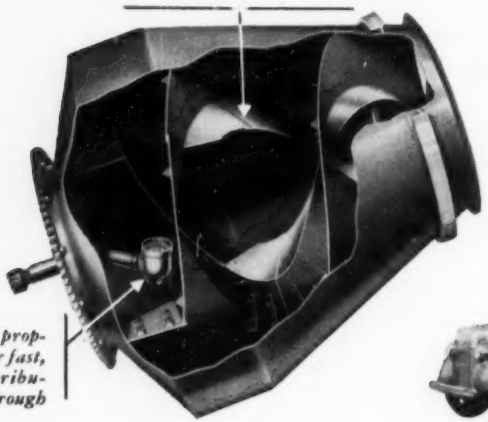
Offices: ALBANY • BIRMINGHAM • BOSTON • CHICAGO • DALLAS • HOUSTON • INDIANAPOLIS • JACKSON, MISS. KANSAS CITY, MO. • NEW ORLEANS • NEW YORK • NORFOLK • PHILADELPHIA • ST. LOUIS • WASHINGTON, D. C.

LONE STAR CEMENT, WITH ITS SUBSIDIARIES, IS ONE OF THE WORLD'S LARGEST CEMENT PRODUCERS . . 15 MODERN MILLS . . 25 MILLION BARRELS ANNUAL CAPACITY

REX MOTO-MIXERS

The Truck Mixers with the "LIVE" Hi-Lo Mixing Action

Large correctly shaped mixer blades are equipped with fillet the entire length of the blade to eliminate build-up and to aid in faster discharging.



Water nozzle properly located for fast, thorough distribution of water through the batch.

Rex has "Live" mixing

Not a "Dead Ending" action

An exclusive feature of the new Rex Moto-Mixer is the up-hill mixing action which has proved an outstanding improvement in truck mixer development. As a result of this action, plus large correctly designed mixing blades, it is now possible to mix low slump concrete—fast.

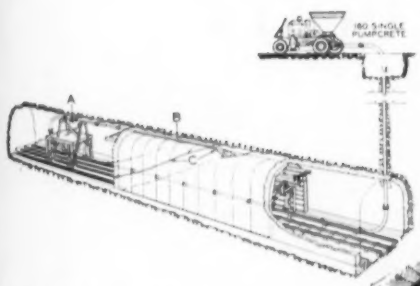
The full volume of the drum is utilized for mixing purposes. This is accomplished by rotating the drum in the discharge direction. The end to end reversing action, spiraling the batch to the top of the drum where it can cascade back to the lower end by gravity, more nearly approaches the efficiency of a large drum stationary plant mixer than anything yet developed in the truck mixer field.

Another important feature is the method of introducing water at a point where it can be distributed throughout the batch quickly. Water is ejected from the nozzle at a point where the mixing blades plow into the batch. The blades form a furrow; and through this furrow, water is more readily and quickly dispersed throughout the entire mix.

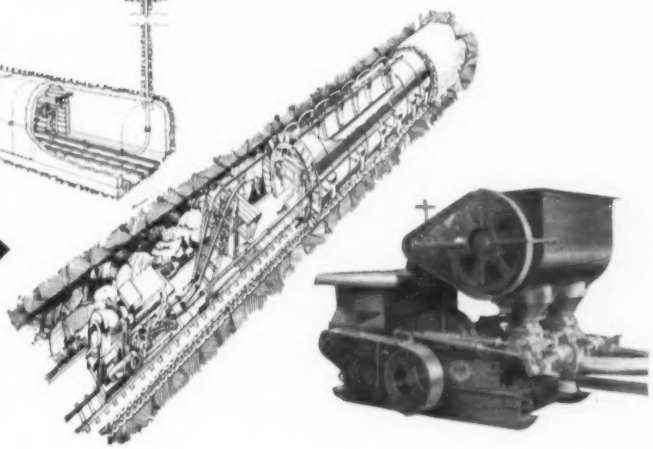
The up-hill mixing action and the more efficient distribution of water throughout the batch are two chief reasons why the new Rex Moto-Mixers produce a more uniform, thoroughly mixed concrete in less time than has formerly been possible with any type of truck mixer. For additional information on the Rex Hi-Discharge Moto-Mixer, send for a copy of Bulletin No. 467. Address Chain Belt Company, 1713 West Bruce Street, Milwaukee 4, Wisconsin.

REX CHAIN BELT COMPANY

TUNNEL LINING WITH THE REX PUMPCRETE



Lining a tunnel with Pumpcrete setup outside. Transporting equipment and labor in the hole are eliminated. Other tunnel operations can be carried on without interference.



Tunnel lining with Pumpcrete setup inside. Pumpcrete is equipped with horizontal remixer which provides storage and prevents segregation. Concrete can be deposited to sidewalls of large tunnels in horizontal layers where this method is desirable.

For tunnels from four feet in diameter up to the largest, the REX Pumpcrete offers a more flexible approach in meeting general tunnel construction problems than any other type of equipment. Its long placing range and general utility for any phase of the job provide the most economical method of concreting on the majority of tunnel projects.

Pumpcretes will handle stiffer concrete with much less loss of slump than other placers operating at an equal distance. In other words, less water per bag of cement is required for a given slump consistency. Discharge to the forms is more readily controlled which minimizes segregation and is another one of the reasons why a Pumpcrete, properly employed, will turn out a better job structurally than any other type of mechanical placer, i.e., by filling the forms tighter with denser, more impervious concrete. The pumping system is also applicable to a much wider variation of procedure in meeting the unusual placing problems that occur on many tunnel lining jobs. It will do a much better job of filling overbreaks; and in tight sections caused by squeezes or falls, where regular arch pipe cannot be installed, it will do a satisfactory job by pumping directly through the form.

Pumpcrete is the answer for most tunnel lining and for many other types of construction work. For detailed information, send for a copy of either TUNNEL LINING WITH THE REX PUMPCRETE or Bulletin No. 466 for general construction work. Address Chain Belt Company, 1713 West Bruce Street, Milwaukee 4, Wisconsin.

of MILWAUKEE REX

OUTSTANDING ROAD BUILDING UNITS

'FLEX-PLANE' Finishing Machines—have finished a complete surface in one pass with a compound screed—3115 lineal feet of 9″ concrete 24 feet wide in one 11 hour day—a world record!

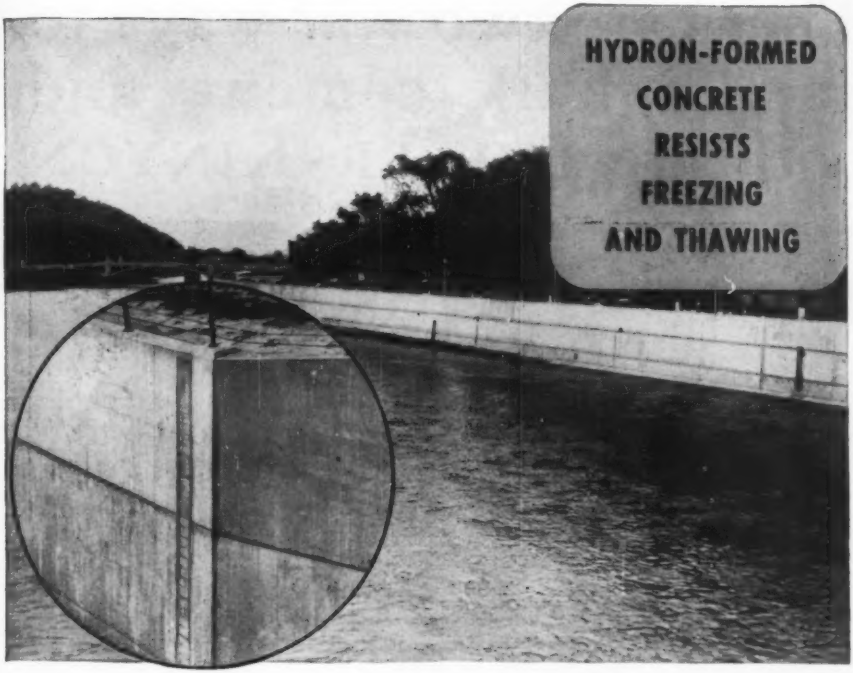
'FLEX-PLANE' Joint Installing Machines for installing joints automatically or semi-automatically—we now provide electric vibration so the concrete around the joint is thoroughly compacted to prevent scaling.

'FLEX-PLANE' Spraying Machines—for curing concrete—an entirely automatic machine—each square yard of surface is double sprayed with proper amount of material.

'FLEX-PLANE' Traffic Line Marking Machines—used in a number of States for dividing or marking traffic lanes.

FLEXIBLE ROAD JOINT MACHINE CO.

WARREN, OHIO, U. S. A.



SUSQUEHANNA FLOOD WALL LIKE NEW AFTER FOUR YEARS OF FREEZE-THAW CYCLES

These are pictures of the Susquehanna flood wall at Binghamton, N. Y. taken last summer. Despite four years of frequent freeze-thaw cycles, the wall still looks like new.

It was cast against Hydron absorptive lining, which reduces by 30% the absorption of water by concrete. Hydron also greatly reduces or eliminates such surface defects as air pits, water channels, and sand streaks.

Tests show conclusively that Hydron produces a case-hardened concrete surface that withstands four times as many freeze-thaw cycles as samples cast against wood.

Before its use on the Susquehanna Flood Wall, or the new MacArthur locks in the "Soo" Canal, or the Norfork Dam, Hydron underwent years of analysis and testing.

Hydron's case-hardening effect is at least one inch deep with a gradual change in the water-to-cement ratio and density from the surface into the bulk concrete.

Made in thin, flexible sheets, Hydron is mounted to forms with rapid-fire staple guns, strips cleanly, needs no tedious finishing.

Send for your copy of informative booklet on Hydron.

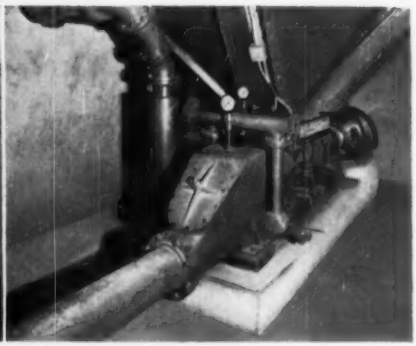
Serving Through Science

UNITED STATES RUBBER COMPANY

1230 Avenue of the Americas • Rockefeller Center • New York 20, N. Y.

There are two *real* ways to handle bulk cement, and both are FULLER-KINYON

Fuller-Kinyon Remote-Control Unloader unloading cement from box car



Fuller-Kinyon Stationary Pump installed in pit underneath tracks unloading cement from hopper-bottom car.

The old saying, "There are no two ways about it," doesn't hold good when applied to the unloading and conveying of bulk Portland cement. Because, there are two **real** ways to do the job, and they're both Fuller-Kinyon. Both systems are efficient and economical of operation, do a quick, clean job of unloading and conveying from box and hopper-bottom cars.

Fuller-Kinyon Remote-Control Unloader—for unloading from box cars, ships and barges. Now used by many ready-mix concrete and asphalt plants, and contractors on highway and dam construction. Any ordinary laborer can operate this equipment with the greatest of ease. Built in different types and sizes for various capacities.

Fuller-Kinyon Stationary Pump—for unloading from hopper-bottom cars. The pump is installed in a pit underneath the tracks. Connection between car and pump is quickly and easily made without dust or loss of material. Built for various capacities from a few tons to 300 tons per hour.

P-77

FULLER COMPANY

CATASAUQUA—PENNSYLVANIA

CHICAGO 3
120 So. LaSalle St.

WASHINGTON 5, D. C.
Colorado Bldg.

SAN FRANCISCO 4,
Chancery Bldg.

Ransome PAVERS

34E SINGLE AND DUAL DRUM

STEADY, ACCURATE WATER SUPPLY to a paver mixing drum is essential if a uniform concrete consistency is to be maintained throughout a job.

RANSOME PAVERS are equipped with a gravity feed water system that is absolutely accurate and trouble-free. It has a non-pressure, syphon type measuring device that measures the batch water accurately, without variation or stoppage of the water flow. Ransome engineers have made this possible by producing a tank which is carefully calibrated, not affected by grades, side inclines, or variations in line pressure. This is one feature of Ransome Pavers. There are many others that are well worth investigating. R6-2

Write for literature

RANSOME MACHINERY CO. Dunellen, N.J.
Subsidiary of Worthington Pump and Machinery Corp.

Other Products: **TRUCK MIXERS, SMALL MIXERS, BIG MIXERS**

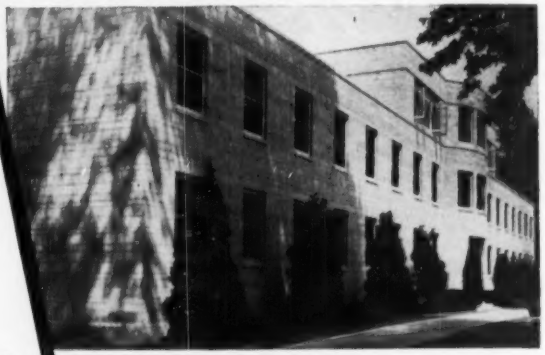
THE MODERN BUILDING MATERIAL

Made on Besser Automatic Plain Pallet Vibrapacs

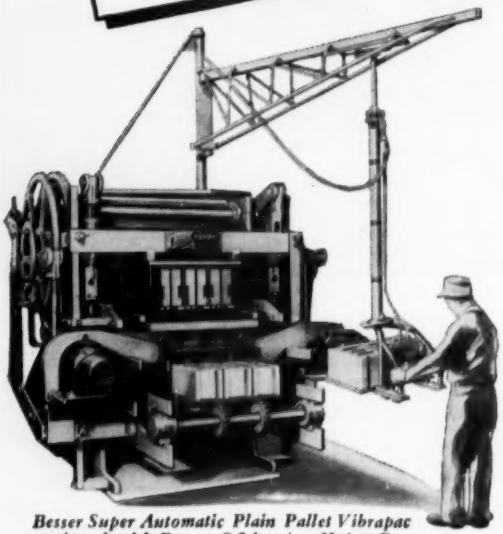
10 *Reasons*

FOR USING BESSER SUPER VIBRAPAC CONCRETE MASONRY BLOCK

1 DESIGN
2 TEXTURE
3 SAFETY
4 DURABILITY
5 SANITATION
6 RODENT RESISTANCE
7 SOUND INSULATION
8 LOW COST
9 LOW UPKEEP
10 TEMPERATURE INSULATION

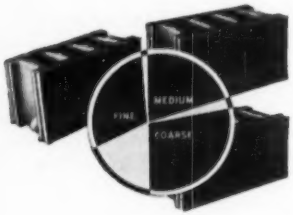


● Concrete Masonry units made on Besser Plain Pallet Super Vibrapacs have become in effect a new building material. Vibrapac control of block density and texture permits manufacture of units adaptable to any construction requirements. The durability and design of Besser made concrete units stand as a building beacon to architects and builders throughout the world.



Besser Super Automatic Plain Pallet Vibrapac equipped with Power Off-bearing Hoist. One man offbears 600 — 8" block per hour. No lifting! Operator merely guides the hoist.

Besser Super Automatic Plain Pallet Vibrapacs make three, 8 x 8 x 16" block at a time on one plain pallet, with a capacity of 600 units per hour. Smaller units in larger multiples on the same pallets. The only stripper concrete block machine that makes all types and sizes of block, brick or tile units on one set of Plain Pallets. Besser Vibrapacs offer the maximum in quality and production at a minimum in operation cost.



BESSER MFG. CO., 902 46th ST., ALPENA, MICH.
Complete Equipment for Concrete Products Plants

Restore the Old-- with Build the New--

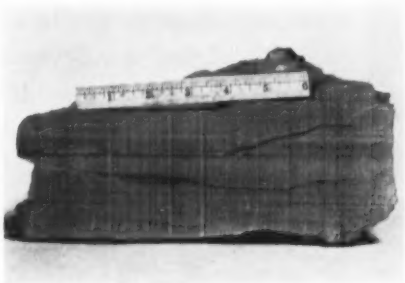
INTRUSION-PREPAKT

INTRUSION

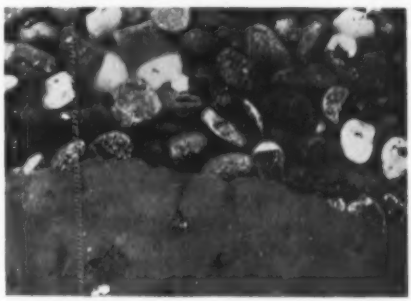
Pressure-filling of spaces within masonry and concrete structures, foundations, and riprap with cement-base binding paste, to solidify the mass.

PREPAKT

Concrete made by packing forms with coarse aggregate and then solidifying by intrusion.



Fine seams in sandstone of ashlar masonry dam filled by intrusion. Mortar joints are likewise solidified.



As mortar is pumped in from below, spaces in prepacked coarse aggregate become completely filled.

APPLICATION

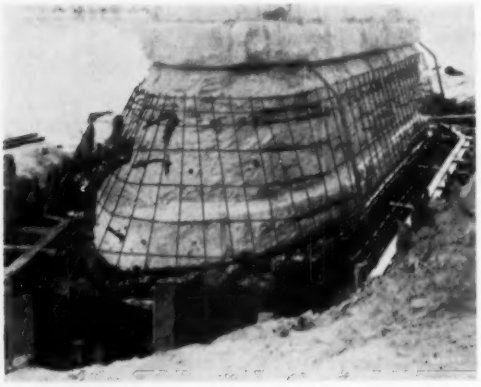
These processes have been successfully used for years on hundreds of projects including

PIERS	WALLS	DAMS
FOOTINGS	VIADUCTS	BREAKWATERS
ARCHES	TUNNELS	ROCK FILLS

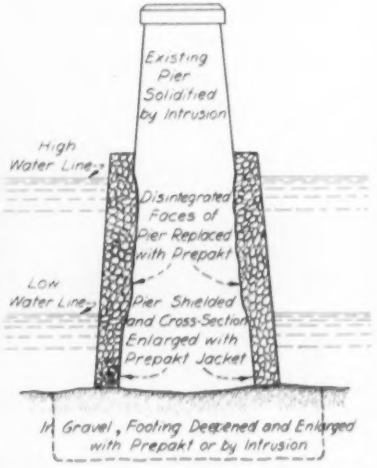
BRIDGE PIERS

Scour of stream beds, abrasion by floating debris and ice, and weathering tend to damage bridge piers by causing inadequate support, unequal settlement, reduction of cross-section, and loss of mortar from joints. Each pier requires special treatment, but all are capable of being restored and protected against attack, through the combined processes of intrusion and prepacking as required. Typical cases are illustrated in the photographs below, together with a drawing showing various means of repair which have actually been used with success. By jacketing with Prepakt, it is possible even to increase the strength and stability of the pier beyond its original capacity, to provide for the heavier loads of modern traffic.

Typical restoration of abutment and wing wall. Bridge seat and disintegrated stones replaced with Prepakt. Interior of masonry intruded down to bed rock.



Scour of river bed exposed timber grillage and piles supporting pier. View shows pier ready for jacketing with Prepakt, after interior has been solidified by intrusion.


Existing Pier Solidified by Intrusion
High Water Line
Disintegrated Faces of Pier Replaced with Prepakt
Low Water Line
Pier Shielded and Cross-Section Enlarged with Prepakt Jacket
In Gravel, Footing Deepened and Enlarged with Prepakt or by Intrusion

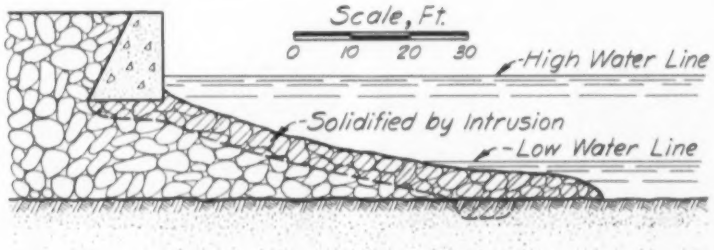
Composite sketch from various jobs, illustrating restoration and strengthening of pier.

BREAKWATERS

Breakwaters, sea walls, rock fills, dams, and gravelly soil bases have been stabilized by intrusion to form monolithic structures. Impervious blankets are provided wherever the full thickness of the structure need not be solidified.

An outstanding example of this type of work is illustrated below. A sea wall subjected to heavy tides, up to 14 ft., and severe storms was periodically washed out and required extensive repairs. To stop this condition and to provide permanent protection to the embankment, the upper portion of the rock fill, 3 to 5 ft. thick, was packed with coarse aggregate and was intruded to form a solid blanket. The cut-off wall was intruded to a depth of 8 ft. The work was accomplished in two construction seasons. During the winter between seasons, severe storms and tides washed out part of the untreated rock fill, but left the treated portion entirely undamaged as shown.

Similarly, an earth fill dam is being treated by flushing out the finer fraction of soil and intruding the gravel.



Section of sea wall, showing stable imprevious blanket and cut-off wall formed by intrusion.

Completed portion of sea wall. Note (in foreground) aggregate used to fill interstices of larger stone, and wooden plugs projecting from vertical intrusion pipes.

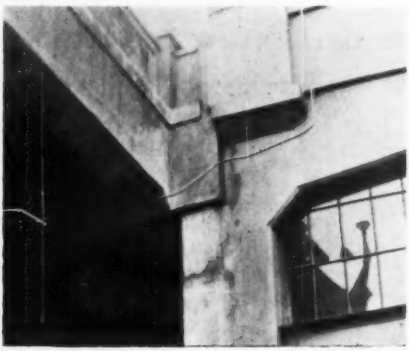
Untreated portion washed out by storms; treated portion undamaged. Construction was completed during following year.

STRUCTURES

Repairs to viaducts, retaining walls, dams, tunnels, and reservoirs have been made by specialized application of the basic processes—intrusion and prepacking. The relatively low shrinkage of Prepakt, due to the initially close contact of pieces of aggregate, minimizes differential shrinkage and assures monolithic action. Durability of structures is being proved repeatedly by actual service conditions. As for ordinary concrete, strengths may be regulated as desired; usually the compressive strength of Prepakt as obtained from test cylinders and cores is of the order of 4000 psi.

An example of structural repair is illustrated below. Binding of the expansion slide-plates of a long viaduct had caused cracking of numerous reinforced-concrete girders at the section of maximum shear. The broken ends were cut away and were rebuilt with Prepakt.

End of girder, broken off by binding of expansion plates, cut away and ready for restoration.



Damaged section replaced with Prepakt. Tight bond to old concrete enables contact surface to carry heavy shear.

For Restoration

INTRUSION-PREPAKT, Inc.

For New Construction

THE PREPAKT CONCRETE CO.

Union Commerce Bldg., Cleveland 14

309 W. Jackson Blvd., Chicago 6

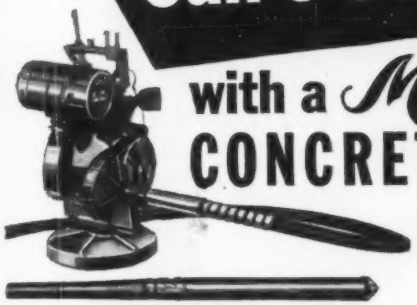
429 Union Station, Toronto, Ont., Can.

Methods and Materials Patented

See Also Preceding Three Pages

Hand Puddling Can't Compete

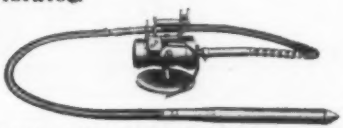
with a Mall REG. U.S. PAT. OFF. CONCRETE VIBRATOR



A Size and Type For Every Job!

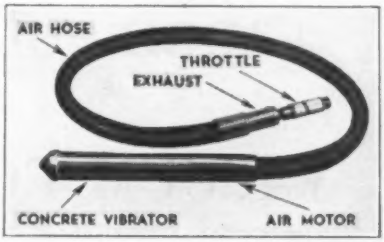
Fewer hands—faster placing—stiffer mix—uniform strength and density—better bond with reinforcement—no voids or honey-combs—water-tight job that permits an earlier stripping of forms—these are just a few of the advantages you get with a Mall Concrete Vibrator. In addition, for their size Mall Vibrators place more concrete than any other vibrator.

When not being used to vibrate concrete, the Mall Vibrator can be adapted to Surfacing, Form Sanding, Wire Brushing, Grinding and Drilling. Attachments for these operations are easily and quickly interchanged with the vibrating element which is constructed of the toughest materials. Tips are welded with special metals to withstand the severe abrasive action which occurs during operation.



MALL 1½ H.P. UNIVERSAL ELECTRIC CONCRETE VIBRATOR. Delivers 9000 vibration frequencies per minute under load. Like other MALL units, it is available with 14 ft., 21 ft., or 28 ft., of flexible shafting. Has low round base mounting and convenient carrying handle.

MALL CONCRETE VIBRATOR POWERED WITH 3 H.P. GASOLINE ENGINE. Delivers 7000 vibration frequencies per minute. Has variable speed, single cylinder, four cycle, air cooled gasoline engine that runs all day on 1½ to 2 gallons of gasoline. The wheelbarrow type mounting provides unusual portability. Attachments can be furnished for CONCRETE SURFACING, WIRE BRUSHING, FORM SANDING, DRILLING in concrete, brick, iron or steel, and SHARPENING TOOLS and BITS.



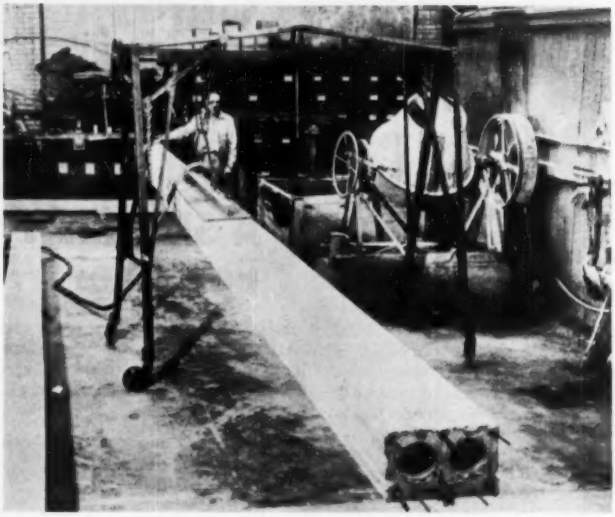
7500 r.p.m. PNEUMATIC MODEL. For placing concrete in tunnels, caissons, and other deep construction. All parts are easily renewable in the field and no special tools are required. Equipped with twist-hand throttle for easy operation.

MALL TOOL COMPANY, 7703 South Chicago Avenue, Chicago 19, Illinois

● 25 Years of "Better Tools for Better Work"

Mall REG. U.S. PAT. OFF. PORTABLE POWER TOOLS

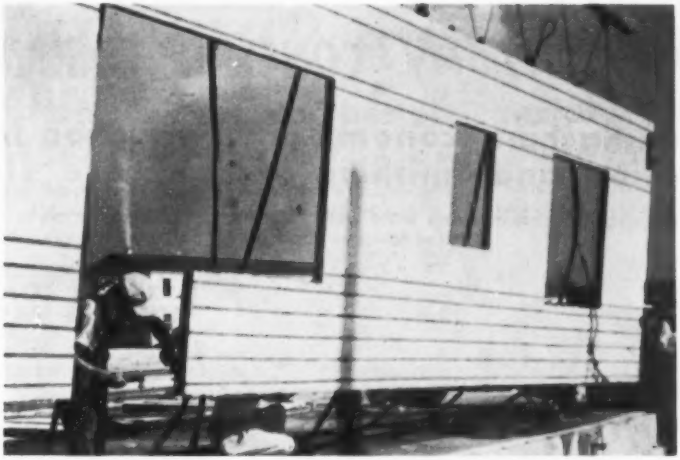
For Mass Production of
PRECAST CONCRETE



PILES FLOORS
BEAMS CRIBBING
WALLS POSTS
LARGE CULVERTS and PIPES

USE
VACUUM CONCRETE FORMS
AND
VACUUM CONCRETE LIFTERS

VACUUM CONCRETE INC.
4210 Sansom St., Philadelphia 4, Pa.



The illustrations indicate the varied uses of the Vacuum Concrete Lifter. This method has been used for moving precast concrete girders, 43 feet long, weighing 4 tons, the day after they were cast.

NOTE: Recognizing the high ethical standards of this publication and the known severity of its advertisement censors, we feel restrained to say only that there is no other process but Vacuum Concrete which does all of the following:

Increases the strength of a plastic mix concrete by an average of 100% in the early curing period and by an average of 50% at the age of 28 days and by an average of 30% at the age of 2 years; and

Provides a method whereby heavy prefabricated concrete members can be handled a few hours after pouring them, by use of the Vacuum Concrete Lifter.

Cast in situ floors:

For quick removal of forms, finishing without delay, reducing overtime, better quality, use the VACUUM CONCRETE PROCESS.

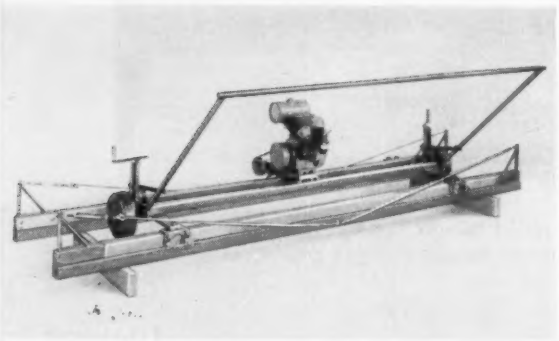
VACUUM CONCRETE INC.

4210 Sansom St. Philadelphia 4, Pa.

Whiteman CONCRETE EQUIPMENT

Provides machine economy and perfection in the preparation and finishing of concrete slabs.

Portable Rodding Machines save time and money by eliminating slow, laborious hand rodding or screeding.

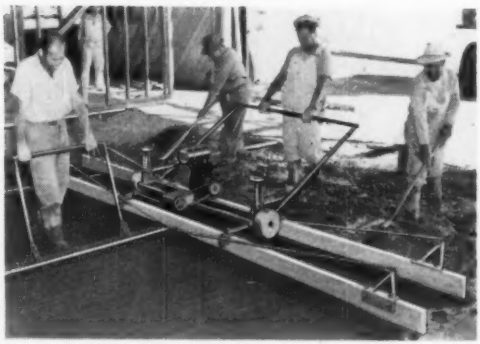


MODEL 44 PORTABLE CONCRETE RODDING MACHINE

Rods the slab to a perfect level and at the same time puddles and vibrates the concrete through the entire depth . . . over the whole area. Thus the slab is thoroughly compacted and the moisture brought to the surface, ready for floating and finishing. For slabs 10 to 20 feet wide. Handles as low as one-inch slump. Operates at 100 strokes per minute on a Wisconsin AB 2.4 HP air cooled engine.

MODEL "RS" Lightweight Rodding Machine

An excellent companion machine for the Model "J" Floating-Finishing Machine. Suitable for slab widths 3 to 12 feet; operates like the larger Model 44. Ideal for sidewalks, driveways and repair jobs. Powered by Briggs and Stratton motor model "N" 1.5 H. P.



HAND GRILL TAMPER

Especially suited for smaller jobs. Lightweight, 5-foot tamper with mesh surface tamps down the larger aggregate, leaving the finer materials on surface. This results in a more dense and compact floor; eliminates the use of dry topping.

***Whiteman* Concrete Equipment pays for itself . . . in lower labor costs . . . finer quality of work.**

Whiteman FLOATING-FINISHING MACHINES produce better concrete surfaces. The ideal method for color application.

Eliminate back-breaking kneework with this modern mechanical troweling machine. It produces a dense, wear-resistant slab, free from riffles and depressions.

MODEL "B" DUAL PURPOSE FLOATING-FINISHING MACHINE

Floating is done with the "Heavi-Duti" 10′ x 18′ flat float trowels. This operation compacts the slab; drives out air pockets; brings up moisture; produces an excellent traction surface for warehouse floors, walks, airplane runways or vehicular traffic. To finish, attach 6″ x 18″ crucible steel trowels. While machine operates control handle is turned to obtain correct trowel pitch. Available with gas engine or electric motor drive. 46″ diameter.

NEW MODEL "J" LIGHTWEIGHT FLOATING-FINISHING MACHINE

Same principle as Model B . . . but smaller capacity. The trowel diameter of 34″ (guard ring 35″) permits operation in small crowded areas and through 36″ doorways. The Model "J" weighs only 105 lbs.; provides easy portability . . . even an inexperienced operator can finish 1000 square feet in fifteen minutes. Available with gas engine only.

METAL AND WOOD SCREED STAKES

For quick placing in any kind of soil . . . used with 2″ x 4″ wood or 2½″ pipe screeds. Final grade obtained by sliding head up or down. The best method of screeding against walls or curb.

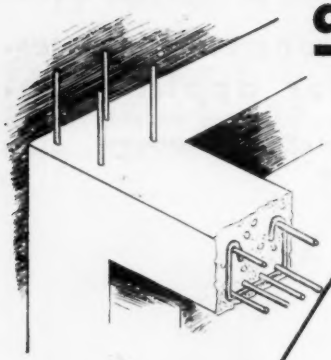
WHITEMAN EQUIPMENT is sold only through established American and Foreign distributors. Write for name of nearest dealer.

CONCRETE EQUIPMENT
Whiteman MANUFACTURING COMPANY
LOS ANGELES CALIF. U.S.A.

Whiteman MANUFACTURING COMPANY
3249 Casitas Avenue
Los Angeles 26, California

HI-BOND*

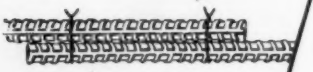
a More Efficient



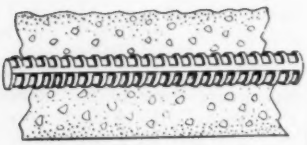
. At Splices

. In Concrete

. For Installations



When HI-BOND bars are spliced, they will develop their tensile strength in splices with a shorter lap than commonly used.



Irrespective of the position in which they are cast HI-BOND bars provide a greater mechanical grip in the concrete and, therefore, offer higher resistance to slippage.

HI-BOND bars stay anchored when they are crossed and wired because the ribs key into each other and hold firmly.

*TRADEMARK-REG. U. S. PAT. OFF.

INLAND HI-BOND

REINFORCING BAR

HI-BOND bars provide the designer with an effective means to utilize more of the potential strength of the steel in reinforced concrete. The design of the helical ribs produces greater bearing area in concrete. This develops higher bond stresses, which permit more efficient load transfer at splices. This is demonstrated in tests reported in research paper 1669 issued by the National Bureau of Standards in their September, 1945, Journal.

These tests prove that HI-BOND bars will develop their tensile strength in splices with a shorter lap than commonly used, whether bars are wired together or spaced apart.

Placing of reinforcing steel is done more speedily and securely with HI-BOND bars. The helical ribs dovetail so well at intersections that the simplest wire tie will hold HI-BOND bars firmly fixed even in difficult wall positions. The saving in time, labor and wire makes possible more economical construction.

With HI-BOND, deflections in beams are reduced, width of cracks minimized, and the possibility of steel corrosion is less. All these advantages result in better, stronger, more efficient installations —no premium is assessed for HI-BOND's improved design—when you specify HI-BOND *you get extra value at no advance in cost.* Inland Steel Company, 38 South Dearborn Street, Chicago 3, Illinois.

REINFORCING BAR

JOHNSON MIXING PLANTS *feature...*

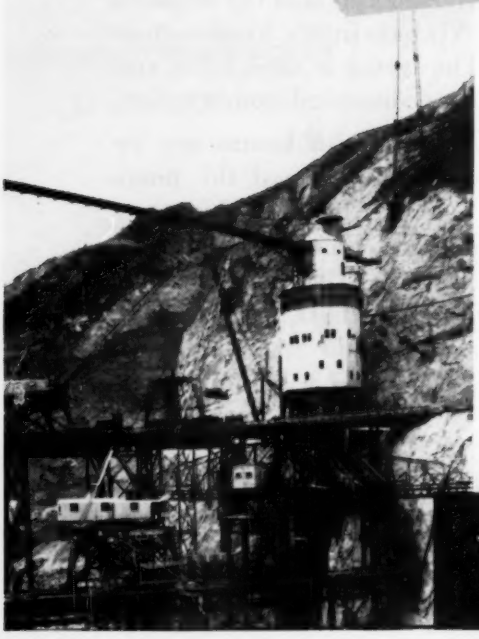
★ **CONCENTRIC ZONE PLANT ARRANGEMENT**
(1)—Permits grouping of 1 to 6 mixers about center discharge point.
(2)—Design affords minimum height of plant.
(3)—Short charging chute eliminates segregation of aggregates and reduces charging time.
(4)—Compact design requires minimum ground area for plant site.

★ **CENTRAL CEMENT COMPARTMENT**
(1)—Steep slopes in cement tank for free, fast-flowing.
(2)—Radial arrangement of aggregate bins around center belt conveyor turnhead, provides flexibility in number and capacity of aggregate compartments.
(3)—Compact arrangement of batchers, with cement flow-stream in center of aggregate flow.

★ **PROVEN AUTOMATIC BATCHER CONTROL**
(1)—Accurate, high-speed, positive batching.
(2)—Moisture compensation for all mixes.
(3)—Instantaneous MIX-SELECTOR SYSTEM.
(4)—Recording of batch weight for each batcher and consistency record of each mixer, on a single chart with timing of each operation permanently recorded.
(5)—Built-in recording chart printer eliminates paper shrinkage problems, and reduces chart costs, being self-printing.

CONTROLLED CONCRETE MIXED BY THESE METHODS, OFFERED ONLY BY

JOHNSON



HAS BUILT THE WORLD'S LARGEST STRUCTURES, FOREMOST BEING GRAND COULEE SHOWN AT THE LEFT—AS WELL AS THESE, AND MANY OTHERS—

FONTANA	SHASTA
NORFORK	RUBY
FRIANT	BONNEVILLE
PENSACOLA	NORRIS
CONCHAS	BOULDER
HETCHHETCHY	CADDOA
OSAGE	PICKWICK
KENTUCKY	MAHONING
HIWASSE	CHEROKEE
TYGART	CHICAMAUGA

SUBMIT YOUR MIXING PROBLEM TO EXPERIENCED JOHNSON ENGINEERS TODAY —WRITE

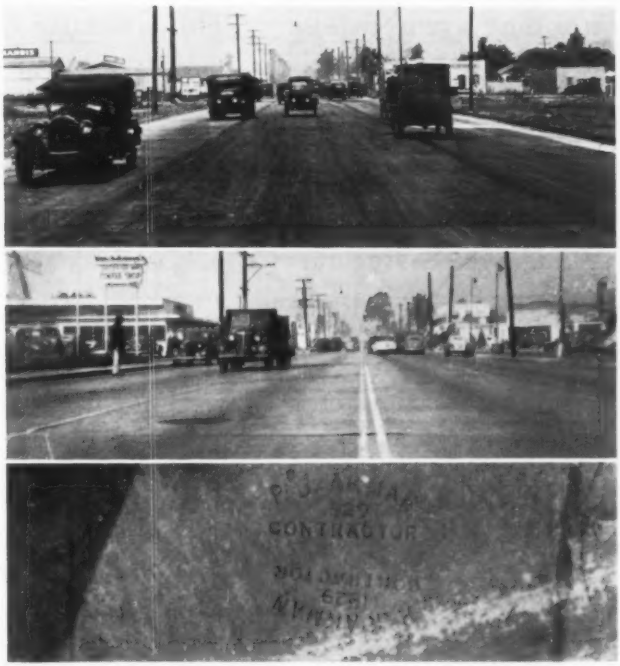
The C. S. JOHNSON CO.
Champaign, Illinois

HUNT PROCESS

THE ORIGINAL CONCRETE MEMBRANE CURE SINCE 1926

EVIDENCE OF THE EFFICIENCY OF HUNT PROCESS CURING

Views of San Fernando Road, (U. S. 99 to San Francisco and North) was cured with "Hunt Process Black" in the summer of 1929. It is one of the most heavily travelled truck and passenger car highways in the City of Los Angeles. The top picture was made July 1929 shortly after pavement was opened to traffic; the center picture was made December 1945.



Contractors stamp, as photographed December 1945, was originally placed in pavement during construction. It shows the complete curing obtained from Hunt Process, with little evidence of surface erosion from 16 years of very heavy traffic. This particular stamp is near a "Stop & Go" intersection where traffic abrasion is particularly severe.

Since 1929 more than 170,000,000 square yards of concrete surface have been cured with Hunt Process Company Curing Compounds, equivalent of more than 13,000 miles of 22-foot concrete.

DISTRIBUTORS

The Lofland Company	Dallas, Texas
The Colorado Builders' Supply Co.	Denver, Colorado
Grace Brothers, Limited	Honolulu, T. H.
The Carter-Waters Corporation	Kansas City, Missouri
The R. C. Hardcastle Company	Oklahoma City, Oklahoma
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American Asphalt Roof Corp.	Salt Lake City, Utah
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Charles R. Watts & Co.	Seattle, Washington
A. E. Foreman & Sons	Vancouver, B. C.

HUNT PROCESS COMPANY

"Spray It & Forget It"

Main Office & Plant
7012 Stanford Avenue
Los Angeles 1, Calif.

HUNT PROCESS CLEAR
HUNT PROCESS BLACK
HUNT PROCESS PIGMENTED-(WHITE)

Kansas City Mo. Plant
Carter-Waters Corp.
2440 Pennway

WHAT PRICE "CONTROLLED AIR" IN CONCRETE?

★

An air entraining agent, whose sole function is to control the air content of the finished concrete, offers definite advantages to concrete engineers and contractors. Besides improving durability and plasticity, the use of such an air entraining agent at the mixer gives the contractor a means of meeting stringent air entraining concrete specifications when using the materials that will be employed on the job.

To the contractor, the use of such an air entraining agent will usually result in a concrete cost actually less than that of untreated concrete because:

1. In making air entraining concretes the specified cement content per unit volume must be maintained. This is usually accomplished by a reduction of the fine aggregate of the mix in proportion to the amount of air entrained.

2. On an average this adjustment of sand per cubic yard of concrete may be taken as 125 pounds.

3. Even with sand costing as little as 60c per ton, a saving over the cost of the air entraining agent is realized.

To the engineer, the use of such air entraining agents will give concrete with *controlled air.*

When such air is held below 5 per cent, and when there is a sufficient reduction of water to

provide an equivalent slump, little or no decrease in strength will result. In fact, in lean mixes (less than 5 sacks per cubic yard) there is usually an increase in strength.

To both engineers and contractors there are many intangible savings which may assume major importance under certain conditions, such as ease of placing, speed of finishing, overcoming irregularities in aggregate gradation, etc.

The above facts have all been proved in the field with Darex Air Entraining Agent, which already has been used in over four million yards of concrete. This material comes in solution form ready to use and lends itself to simple proportioning at the mixer.

★ ★ ★

Use DAREX AEA with confidence wherever air entraining concretes are specified.

Write for booklet and manual on the use of DAREX AEA.

DEWEY AND ALMY CHEMICAL COMPANY

Department T - - Cambridge 40, Massachusetts

THE FOUNDATION OF

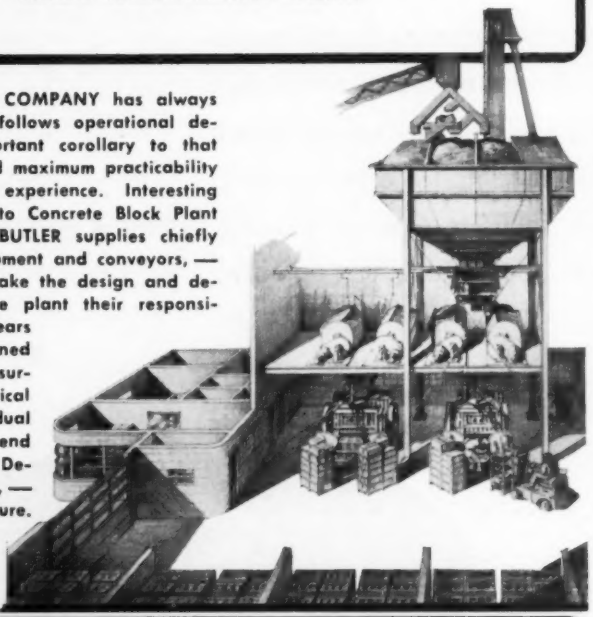
BUTLER ENGINEERED DESIGN

IS THE WIDE EXPERIENCE OF MANY YEARS

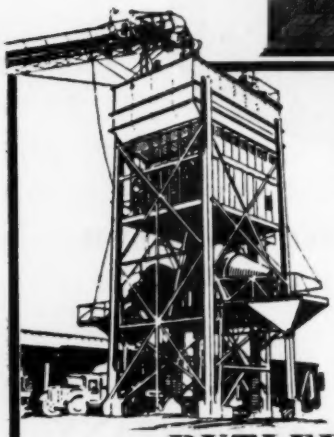
. . . .

ITS PRODUCT IS

HIGHER EFFICIENCY, GREATER PRODUCTION,- AT LESS ULTIMATE COST



While the BUTLER BIN COMPANY has always held that manufacture follows operational design, there is an important corollary to that principle: Ingenuity and maximum practicability in design must follow experience. Interesting is the approach given to Concrete Block Plant construction. Actually, BUTLER supplies chiefly the bins, batching equipment and conveyors, — yet BUTLER Engineers make the design and development of the entire plant their responsibility. And the many years of wide experience gained in this field bring an assurance of the most practical solution to every individual problem. You may depend upon Butler Engineered Design, — and of course, — upon Butler manufacture.



BUTLER READY MIXED CONCRETE PLANTS represent not only the BUTLER ideal that the making and fabrication of materials handling equipment should be flawless, — but quite as importantly, — that the design of such equipment should be superior in affording greater daily output and maximum efficiency to the operator. That is why consultation with a Butler Engineer will prove profitable.

★

Ready Mixed Concrete Plants
Central Mixing Plants
Concrete Block Plants
Bulk Cement Plants
Batching Plants
Carscoop

BUTLER BIN COMPANY

WAUKESHA, WISCONSIN

Calcium Chloride Curing Provides Maximum Hardness for Concrete, According to Abrasions Tests at National Bureau of Standards

Greater concrete hardness is provided by calcium chloride curing than by other methods of curing tested, according to tests of resistance to abrasion made at the National Bureau of Standards by H. C. Vollmer and reported in his paper appearing in the proceedings of the 24th Annual Meeting of the Highway Research Board.

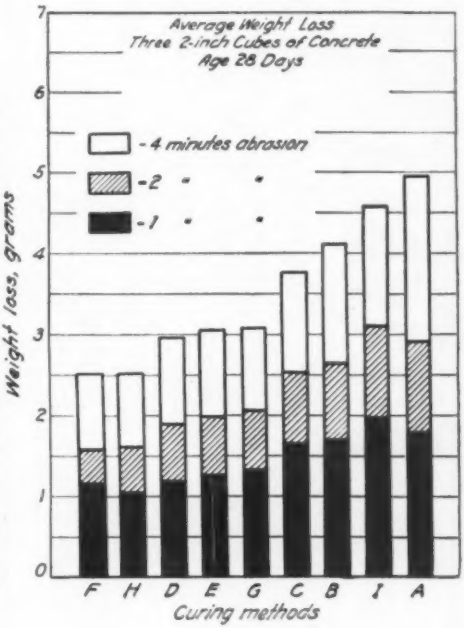
These results are most important to concrete engineers interested in the enormous building program ahead in which emergency restrictions no longer apply and the best of materials and methods of construction will be used.

Mr. Vollmer describes the results of his tests of resistance to abrasion of concrete specimens cured by various means, as follows:

"Tests of the resistance to abrasion of the top surface (cured surface) of the specimens indicated that the use of 1½ *per cent calcium chloride integrally* (equal to 1½ lbs. per bag of cement in the mix) and 1½ *lbs. per sq. yd.* applied to the surface as soon as the bleeding water disappeared (no burlap) and all specimens cured by employing the surface application of *calcium chloride* whether in conjunction with burlap or not, *resulted in a higher resistance* than specimens cured with wet burlap applied for three days; however, the wear resistance of specimens with *liquid curing membranes or with no curing were somewhat less* than specimens cured with wet burlap applied for three days."

Copies of Mr. Vollmer's complete report will be sent on request.

RESISTANCE TO ABRASION



F—1½ lbs. calcium chloride per sq. yd. surface and 1½% calcium chloride integral.

H—2 lbs. calcium chloride per sq. yd. surface with 18 hrs. wet burlap.

D—2 lbs. calcium chloride per sq. yd. surface (no burlap).

E—2% integral calcium chloride with 18 hrs. wet burlap.

G—3 days wet burlap.

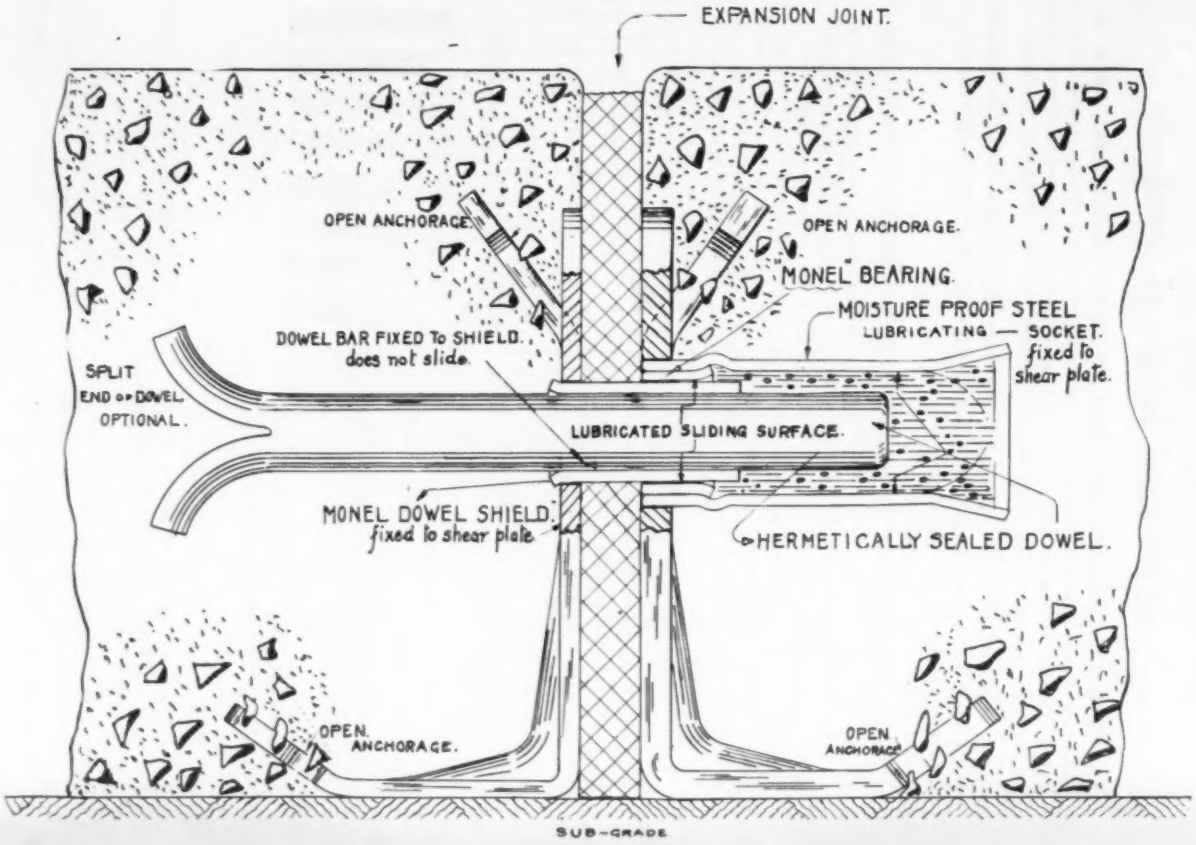
C—Surface coating (Membrane C).

B—Surface coating (Membrane B).

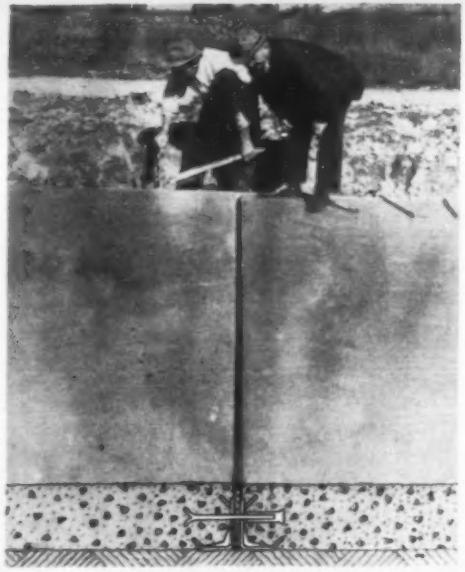
A—Surface coating (Membrane A).

I—No curing.

CALCIUM CHLORIDE ASSOCIATION, 4145 Penobscot Bldg., Detroit 26, Mich.



Scientists nev-



Scientists never gave up their determination to produce an Atomic Bomb.

Engineers never gave up their determination to secure a practical load transfer element for expansion and contraction joints for modern concrete highways and airports.

Heltzel engineers have the answer to every single expansion joint problem involving load transfer elements.

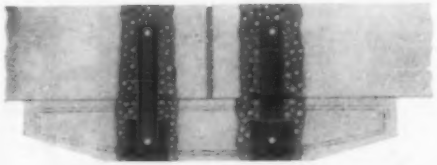


Fig. 4—Helco Tension Dowel and Expansion Joint Beam for Highways

Are you interested in permanently lubricated dowels, frictionless dowels—submerged load transfer elements, non-corrosive dowels, hermetically sealed dowels or tension dowels.

HELTZEL STEEL FORM & IRON CO.
WARREN, OHIO, U.S.A.

Planned Concrete Form Work

Requires Consideration of:

BREAKDOWN OF MONOLITHS

FACTORS INFLUENCING FORM DESIGN

CONCRETE PRESSURES

CLASS OF FORMWORK

BALANCED DESIGN OF FORM MEMBERS

CONCRETE PLACEMENT METHODS

FORM STRIPPING PROCEDURE

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SELECTION OF TIE AND ANCHORAGE UNITS

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MANUFACTURING SINCE 1911

PORTLAND CEMENT DISPERSION
and ADSORPTION OF CALCIUM LIGNOSULFONATE
with POZZOLITH

Pozzolith, which applies the well known principle of dispersion to Portland cement, was developed by Master Builders Research Laboratories following their discovery of the cement dispersing action of calcium lignosulfonate.

Cement dispersion reduces the water-cement ratio required for a given workability, increases strength, and greatly improves durability as well as having a number of other beneficial effects. 1

A study of cement dispersion recently reported by Fred M. Ernsberger and Wesley G. France of Ohio State University provides evidence of the actions and results of cement dispersion as well as a mechanism by which it may be accomplished. 2

This article —"Portland Cement Dispersion by Adsorption of Calcium Lignosulfonate", establishes two points: (A) that cement when placed in water normally flocculates and is dispersed by the action of calcium lignosulfonate so that the full cement area is subject to hydration. Through microscopic examination this study established the fact that cement particles clump together or flocculate when placed in water; that there are many more discreet particles, particularly small particles when the water medium contains about 0.1% of calcium lignosulfonate. Application of Stokes' law to falling cement particles in a water medium provided a method for measuring the degree of dispersion with and without calcium lignosulfonate. The comparison is shown in the following table. (B) that the mechanism by which cement dispersion is accomplished is mutual repulsion due to the adsorption of like electric charges on the surface of the cement particles. Dispersion of small particles in aqueous suspensions is usually due to mutual repulsion resulting from particles bearing a like electrostatic charge. Use of the following technique determined that cement dispersion is due to the same principle: By examining under the microscope the electrophoretic migration of suspended cement particles, it was observed that when suspended in *distilled water* there was no movement of the particles toward either electrode; that when suspended in *calcium lignosulfonate* solution there was a readily observable movement toward the anode.

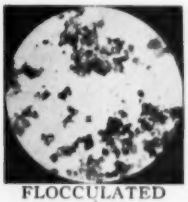
Specific Surface of Portland Cement Suspended in Water, with and without Calcium Lignosulfonate

Ca Lignosulfonate Adsorbed, G./100 G. Cement	Av. Sp. Surface, Sq. Cm./Gram	Apparent % Increase
Type I Cement: 90% Passing 325-Mesh Sieve; Sp. Surface in Kerosene, 1690 Sq. Cm./Gram		
None	1150	..
0.2	(1505)	31
0.3	1570	37
0.4	(1685)	47
0.5	1705	48

Calcium lignosulfonate is a colloidal electrolyte which ionizes in solution to give metallic cations and lignosulfonate anions. The study showed that it is the adsorption of the lignosulfonate anions by the cement particles that gives them the negative charge. Having a like electrostatic charge, the particles repel each other, resulting in cement dispersion.

1 A.C.I. Journal Feb. 1943, pages 358-362 for fullIer description and effects of cement dispersion.

2 Industrial and Engineering Chemistry — June 1945, pages 598-600.

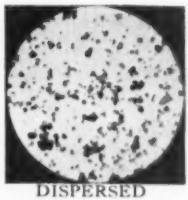


FLOCCULATED

PHOTOMICROGRAPHS OF CEMENT SUSPENSIONS

Illustrating Dispersion

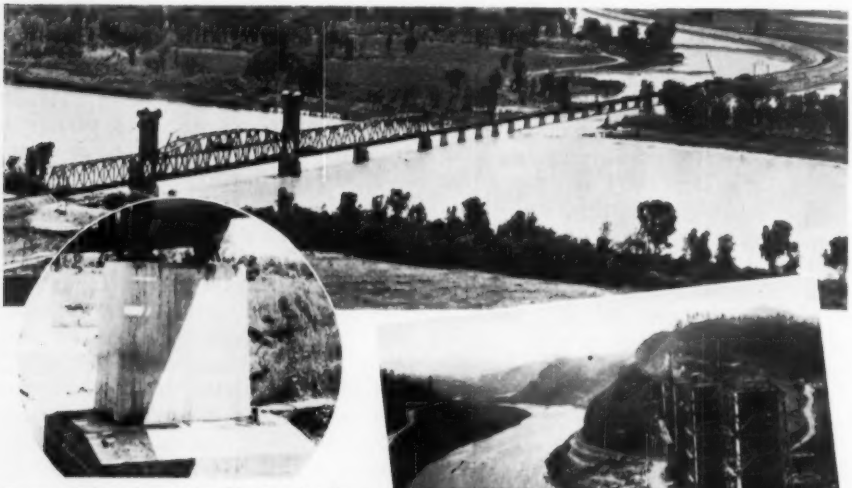
Magnified 200 times



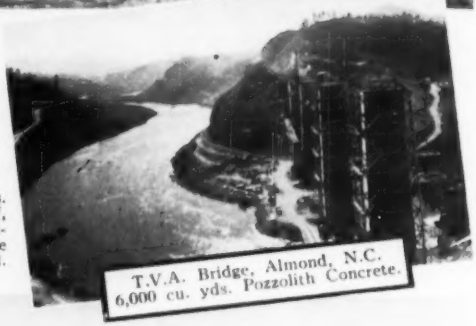
DISPERSED

Showing how cement particles clump together or flocculate when placed in water. By flocculating, many of the cement particles are only partly wetted and some are not wetted at all. Furthermore, when cement flocculates it bottles up a small amount of water inside the clumps.

Cement dispersion causes the clumps to break up and particles to separate. The first result of cement dispersion is that many more of the particles get wetted all over. This puts more of the cement to work. In addition, by breaking up the clumps the entrapped water is released and becomes part of the mixing water.



Truman Bridge, Kans. City, Mo. Consltg. Engrs. — Howard, Needles, Tammen & Bergendorff, Kans. City, Mo. Sub-Structure. Contrs.—Massman Const. Co., Kans. City, Mo. Super-structure Contrs.— American Bridge Co., Chicago, Ill. Pozzolith Concrete.



T.V.A. Bridge, Almond, N.C. 6,000 cu. yds. Pozzolith Concrete.

Fort Stanton Park Reservoir, Washington, D.C. Design Engrs.— Whitman, Requardt & Associates, Baltimore, Md. Contrs.— Baltimore Contractors, Baltimore, Md. 6,500 cu. yds. Pozzolith Concrete.

AUTHORITATIVE TESTS
show
POZZOLITH gives VASTLY GREATER DURABILITY with INCREASED STRENGTH

In an exhaustive investigation employing a range of cements typical of all U. S. Cements — (Full reports available for examination.)

1. Pozzolith increased DURABILITY —

SEA WATER — after 142 Cycles of Outdoor Freezing and Thawing

Nominal Cement Factor — Sacks		
6	Plain Concrete — of 39 beams tested	15 survived
6	Pozzolith " — " 77 " "	76 survived

FRESH WATER — In Laboratory — Freezing and Thawing

		After Average No. Cycles	Average Durability Factor	Average Change in Dynamic E
4½	Plain Concrete	49	17	—31
4½	Pozzolith "	139	63	—21
6	Plain "	50	20	—32
6	Pozzolith "	176	80	—10

2. Pozzolith increased STRENGTH —

	6 Sack Mixes	4½ Sack Mixes
3-day Compressive	50%	70%
28-day "	12%	25%

3. Pozzolith cut BLEEDING 55% 60%

4. Pozzolith cut WATER-CEMENT Ratio 14% 15%

TEST REPORTS FROM OTHER SOURCES

Strengths at Later Ages

Pozzolith concrete, cement dispersed, continues to increase in strength over the long periods. The following compressive strength tests on Pozzolith concrete employed on Ansco Buildings, Binghampton, N. Y., are reported by Cornell University.

C/F	W/C	SLUMP	COMPRESSIVE STRENGTH — DAYS					
			3	7	28	90	6 mos.	12 mos.
5.85	5.69	5"	3115	4440	5765	7580	8320	8370

Tests of Pozzolith Concrete made and reported by Milton Hersey Co., Ltd., Montreal, Canada, industrial chemists, engineers and inspectors are:

Lbs. Cement	Lbs. Water	Slump	COMPRESSIVE STRENGTH — DAYS				
			3	7	14	28	12 mos.
355	273	4"	1115	1575	1995	2335	3285
680	282	4"	3750	4265	4910	5370	7070

Tests of Pozzolith Concrete made and reported by Smith-Emery Co., Los Angeles, inspecting and testing engineers are:

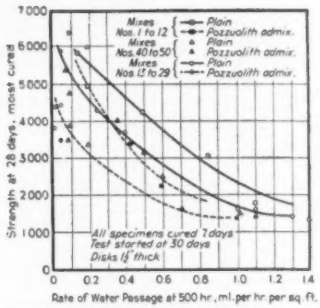
C/F	W/C	SLUMP	MODULUS OF RUPTURE — P.S.I. — DAYS					
			3	7	28	60	6 mos.	12 mos.
5.50	5.44	2.25"	515	537	658	716	750	791
4.95	6.00	2.00"	507	517	625	680	708	735

Pozzolith Reduces Bleeding

ADDITION	PERCENT BLEEDING			
	Cement Factor—4.5 sacks/cu. yd.		Cement Factor—6.0 sacks/cu. yd.	
	Cement Y	Cement Z	Cement Y	Cement Z
None	100	100	100	100
Air-entraining Agent A	52	49	42	34
Air-entralning Agent B	59	56	49	49
Pozzolith	55	57	25	20

POZZOLITH DECREASES PERMEABILITY

"With the same water-cement ratio, a change in the water percentage (space occupied by water at the time of mixing) by the use of water-reducing agents or by varying the aggregate characteristics will have little effect upon watertightness when measured by low-head (liquid) flow. This leakage varies with the strength of the concrete. In the tests shown herein, a commercial admixture "Pozzolith" permitted a reduction in mixing water; this permitted a reduction in cement content for the same strength. An improvement was noted in the capillary resistance of the concrete containing the Pozzolith (even at the same strength)". [1]


Mixes Nos. 1 to 12
Plain
Pozzolith admix.
Mixes Nos. 40 to 50
Plain
Pozzolith admix.
Mixes Nos. 13 to 29
Plain
Pozzolith admix.
Strength at 28 days, moist cured
All specimens cured 7 days
Test started at 30 days
Disks 1/2" thick
Rate of Water Passage at 500 hr., ml. per hr. per sq. ft.

PLOTTED SUMMARY OF TESTS[1]
Relationship between 28-day strength and rate of water loss at 500 hr. Since all tests were made from concrete of plastic consistency, these data indicate an improvement in resistance to capillary movement of water with increase in strength. Pozzolith, used because it reduced water requirement, made an added improvement.

POZZOLITH INCREASES BOND STRENGTH

Bond Strength of Concrete To Steel
Lbs./Sq. In. of embedment at Slip of 3 x 10–

	CEMENT Y		CEMENT Z	
Addition	4.5 sks/cu. yd.	6.0 sks/cu. yd.	4.5 sks/cu. yd.	6.0 sks/cu. yd.
None	354	412	423	470
Air-entraining Agent A	270	412	300	473
Air-entraining Agent B	230	355	342	496
Pozzolith	430	582	302	638

POZZOLITH HOLDS THE CONCRETE IN A PLASTIC STATE WHILE THE CONCRETE IS STANDING OR IN TRANSIT

"Concrete aggregates were shipped by rail from Portland to a point on Clatsop spit about 3½ miles from the end of jetty, where the concrete was mixed. Transportation to the work was in 4-cu. yd. bottom dump buckets, four being placed on each of 3 standard gauge railroad flat cars.

The grading of aggregates and the mix were in accordance with standard government specifications, and provided for at least 4½ sacks of cement (Fed. Spec. SS-C-206) per cu. yd. The contractor was allowed to use an admixture of Pozzolith, at his own expense, to provide greater workability, as the long journey over the rough railroad track otherwise made it difficult to dump the concrete on arrival at the work. The finished concrete structure has now been through three severe winter seasons and does not show any deterioration or cracks." [2] See photograph opposite page.

1 Methods for Measuring the Passage of Water through Concrete. W. M. Dunagan, Associate Professor of Theoretical and Applied Mechanics, Iowa State College, Ames, Iowa. Proc. ASTM Vol. 39, 1939, pages 866-880.

2 Pacific Builder and Engineer, April 1944 issue, Pages 41-44 on "Placing a Heavy Concrete Terminal on the South Jetty of the Columbia River" by R. E. Hickson, Head, Rivers and Harbors Division, Portland District, U. S. Army Engineers.

AIR ENTRAINMENT

. . . In Structural Concrete

Pozzolith in addition to its basic action of cement dispersion, automatically entrains 3% to 4% total air which is the optimum amount with respect to increased durability and strength and improvement of the other properties of concrete, including economy These advantages are obtained with Pozzolith *without the need of constant watching on the job.*

The authoritative tests previously referred to, made by the Nation's highest testing authority, show that with Pozzolith, using 13 different cements in 6 sack and 4½ sack mixes, the total average air, regardless of slump, ranged from 3.07% to 3.75%.

. . . In Flat Slab Construction

For pavement in freezing climates some believe that air contents greater than 3% would be desirable, sometimes even if it means the sacrifice of other desirable properties of the concrete. These higher air contents can be entrained by the use of our HP-7 with no increase in cement content, yet without any loss in strength or other essential value as has been demonstrated in highway pavements and on aircraft landing strips laid with HP-7 in Ohio and Michigan. 1

HP-7 is a combination of the cement dispersing agent calcium lignosulfonate and the effective wetting agent, sodium lauryl sulfate (Orvus), and successfully meets current air-content and durability requirements with increased strengths due to the more effective utilization of the cement.

← Concrete Terminal on the South Jetty of the Columbia River. U. S. Army Engineers. Pozzolith Concrete.

Gulf, Colorado & Santa Fe Railway Underpass, Ft. Worth. Tex. Engrs.— Private Plans. Gen. Contrs.— Martin & Grace, Dallas, Tex. Pozzolith Concrete furnished by Ft. Worth Sand & Gravel Co,. Ft. Worth, Texas.

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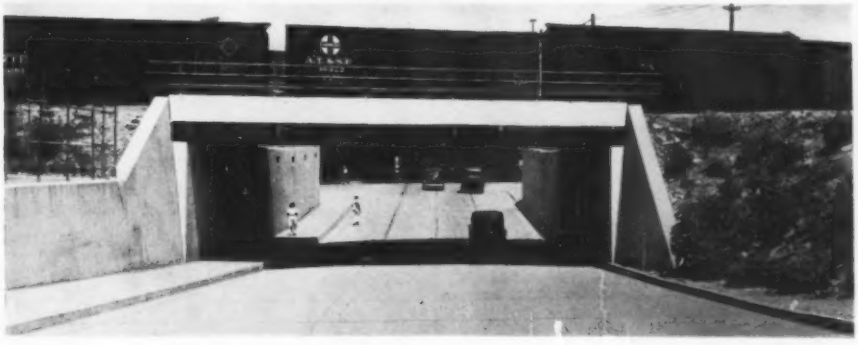


1 A.C.I. Journal, Feb. 1944, pages 358-361 for data on HP-7.

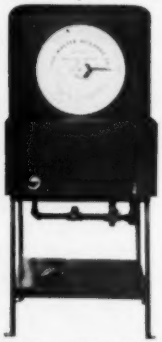
M. B. AUTOMATIC POZZOLITH DISPENSER

Leading authorities agree that to obtain the best results admixtures should be added at the mixer and not interground in the cement. Reference is made to —

J. W. Kushing — Roads and Streets, Dec. 1941.
F. H. Jackson — Principal Engineer of Tests, Div. of Physical Research, Public Roads Administration — Public Roads, Vol. 24, No. 4, Pages 108, 109.
Chas. E. Wuerpel — Engineer-in-Charge, Central Concrete Laboratory, Corps. of Engineers, U. S. Army, Mt. Vernon, N. Y.— 2nd Interim Report on "Concrete Research"— Technical Reports, Pages 47 to 51, July 1945.
Chas. E. Wuerpel — Crushed Stone Journal, June 1945, Page 13.
Chas. E. Wuerpel — A.C.I. Journal, Sept. 1945, Page 81.
Stanton Walker — Engineering Director National Sand & Gravel Ass'n. and National Ready Mixed Concrete Ass'n.— Concrete, Nov. 1945, Page 19.

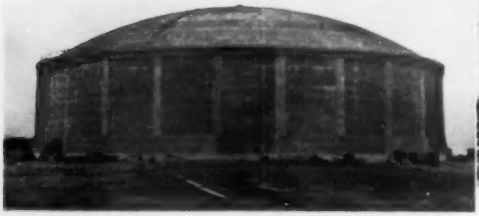
The use of Pozzolith by ready-mixed concrete companies as well as on large jobs has been greatly simplified with the development of the M. B. Automatic Pozzolith Dispenser. In addition to insuring accuracy in measurement, this dispenser cuts cost by saving labor and speeding the operation.

Where ready-mixed plants are equipped with only one cement bin and the handling of special cements involves extra expense, or where they desire to use only one type of cement, the Automatic Pozzolith Dispenser is of special value since Pozzolith added to normal Portland cement produces concrete with high-early strength, decreased permeability and other special properties.

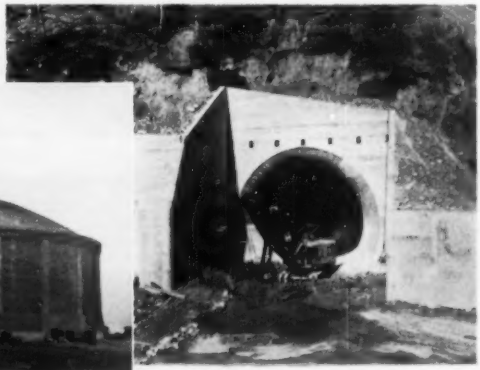


Dispensing Pozzolith in *liquid* form, the operator delivers the correct amount required for a batch by simply setting the control dial and pushing an electric button.

M. B. Automatic Pozzolith Dispensers are now in operation in about fifty ready-mix and batching plants.[1] This dispenser complies with existing specifications for accuracy and may be had with autographic recording devices.



4,750,000 gal. Water Storage Tank, Great Falls, Mont. Consltg. Engrs. & Archts.— Corwin & Co., Inc., Gen. Contrs.— Dudley-Anderson Co., 3,600 cu. yds. Pozzolith Concrete. See Eng. News Record, Oct. 4, 1945, pages 108-113.



Tennessee Valley Authority — S. Holston Tunnel, Bristol, Tenn. 33,000 cu. yds. Pozzolith Concrete.

[1] Pit and Quarry, June 1945, Pages 99-102,— Pit and Quarry, Aug. 1945, Pages 119, 120 and Rock Products, Nov. 1945, Pages 68-70 for experiences of plants using M. B. Pozzolith Dispenser.

BENEFITS OF CEMENT DISPERSION IN CONCRETE FLOOR CONSTRUCTION, MASONRY MORTARS, EQUIPMENT GROUTING & CONCRETE REPAIR

Concrete Floor Wearing Surfaces: Metallic aggregates (Master Builders Metallic Hardner and Metalicron) have been widely used for industrial concrete floor wearing surfaces for over thirty years. The useful life of this type of floor was vastly increased through the development in 1938 of Masterplate, a combination of scientifically graded metallic aggregates with cement dispersion. One hundred and twenty pounds of this wear-resisting metal aggregate is now incorporated easily with low water cement ratio where formerly the limit was approximately 40 lbs. per square. This heavily armored surface vastly increases the durability of concrete floors,[1] so that Masterplate concrete floors have been adopted in many industries where previously concrete floors had not been adequate.

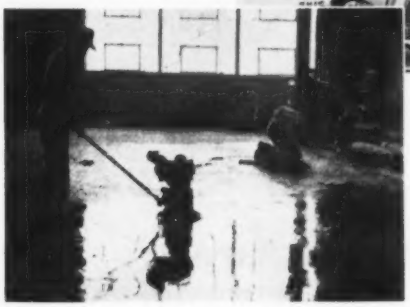
Furthermore, a Masterplate floor provides for static dissemination and reduces sparking to a minimum. This has resulted in the wide use of Masterplate in areas requiring maximum safety, particularly in industries with explosive processes, and in Army and Navy magazines and storehouses.[2] Masterplate floors meet the safety requirements of leading fire insurance companies.[3]

1 National Bureau of Standards Research Paper No. 1252, November 1939.

2 Construction Methods, Sept. 1945, Page 98 for use of Masterplate in rocket plant as protection against sparking.

3 "Handbook of Fire Protection"— 1941 Edition, National Fire Protection Ass'n.

The Aluminum Co. of Canada, Ltd., Arvida, Que. Plans by Owners Engrs. Contrs.— Foundation Co. of Canada, Ltd., Montreal, Que. 300,000 cu. yds. Pozzolith Concrete.



LESS HAZARDOUS AREAS in hazardous buildings are given monolithic metallic finish on concrete floors as protection against sparking. In accordance with specifications, dry mixture consisting of metal aggregate (free from non-ferrous metal particles), cement dispersing agent and pozzuolanic material capable of combining with free lime to form water-insoluble compound is applied to freshly screeded concrete surface at 90 lb. per 100 sq. ft., and floor is finish-compacted by motor-driven disk-type float.

Masonry Mortar—Cement Dispersion is available for masonry mortars in the product "O.M." (Omicron Mortarproofing) which provides increase in water retentivity, reduction in water content, and increase in bond strength. These benefits are direct remedies for early mortar shrinkage, the principal cause of leakage in brickwork and other masonry. 1

Building Restoration with Embeco at Diamond Match Co., Oswego, N. Y.

Equipment Grouting — Embeco is a dry powder composed principally of the cement dispersing agent, specially prepared metallic aggregate and reagents to promote oxidation and strength. Added to concrete or mortars it eliminates or compensates for all shrinkage. Where used for equipment grouting the action of Embeco is so controlled as to produce a slight expansion, thereby assuring a solid lasting contact between foundation and machine. 2

Embeco overcomes the natural shrinkage in patching concrete. Because of this fact it is used in the rebuilding of spalled and eroded concrete surfaces. 3

1 The Architectural Record, November, 1935, pages 347-351 for reference on "The Principal Cause of Leaky Brickwork".

2 Steel — June 4, 1945, Pages 118-120 for use of Embeco in grouting steel mill equipment.
Railway Age, May 16, 1936, Pages 792-794 "Shift 90 ft. Towers Between Trains . . ."
Railway Age, Jan. 9, 1937, "Care and Precision Feature Rebuilding of Tunnel Floor . . . Non-Shrinking Grout Holds Tie Blocks Securely".

3 Contractors and Engineers Monthly, Dec. 1936 —"Defective Pavement Effectively Repaired".
Railway Engineering and Maintenance, Feb. 1937, "Timely Repairs Save Concrete".
Engineering News-Record, Feb. 25, 1937 —"Heated Air Space Eliminates Frost Destruction in Tank Walls".
Water Works and Sewerage, July 1939 —"A Successful 'Face Lifting' Operation . . .".
Concrete, Dec. 1939 —"Restoring Surface of Huge Concrete Bins Spalled by Elevator Fire".

May we take this opportunity to invite members of the Institute to visit our Research Laboratories and to call on us for additional data on the products discussed here and other Master Builders products and services.

Services available through our representatives located in principal industrial centers in the United States, in Latin America and in several foreign countries.

Metropolitan Life Insurance Park-Fairfax Project, Alexandria, Va. Archt.—Leonard Schultze & Assoc., Alexandria, Va. Contr.—Starrett Bros. & Eken, New York, N.Y. 20,000 cu. yds. Pozzolith Concrete and "O.M." for 30,000,000 brick.

Compressors grouted with Embeco at East Ohio Gas Co., Cleveland, Ohio.

THE MASTER BUILDERS COMPANY

CLEVELAND 3, OHIO **TORONTO, ONTARIO**

The AMERICAN CONCRETE INSTITUTE

is a non-profit, non-partisan organization of engineers, scientists, builders, manufacturers and representatives of industries associated in their technical interest with the field of concrete. The Institute is dedicated to the public service. Its primary objective is to assist its members and the engineering profession generally, by gathering and disseminating information about the properties and applications of concrete and reinforced concrete and their constituent materials.

For nearly four decades that primary objective has been achieved by the combined membership effort. Individually and through committees, and with the cooperation of many public and private agencies, members have correlated the results of research, from both field and laboratory, and of practices in design, construction and manufacture.

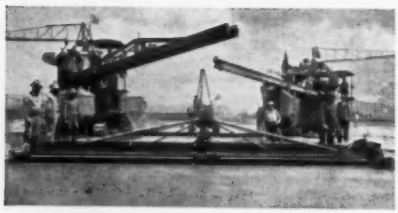
The work of the Institute has become available to the engineering profession in annual volumes of ACI Proceedings since 1905. Beginning in 1929 the Proceedings have first appeared periodically in the Journal of the American Concrete Institute and in many separate publications.

For further information about ACI Membership and publications address:

Secretary, AMERICAN CONCRETE INSTITUTE
New Center Building, Detroit 2, Michigan, U. S. A.

MASTER VIBRATORY FINISHING SCREED

- **How it provides accurate strike-off and compaction in a single easy operation.**
- **How it finishes over 6,000 sq. ft. per hour.**



Note. Hand-drawn Vibratory Finishing Screed, in the main photograph, control throttle at right end provides easy regulation of vibrating speed. The small illustration above shows a 26′ Vibratory Finishing Screed working in California on a 25′ pavement strip of 11″ thickness . . . and finishing strip in one pass. Pavers are 34-E single and 34-E dual drum.

Master Vibratory Finishing Screed works only part time to take care of pavers' output. Moves forward at 6′ per min. One screed should handle two dual drum pavers for maximum efficiency on this type of strip. Reports indicate that finishing done by Master screed equals two screed type tamper-equipped finishing machines obtaining better compaction and requires less cement finishers.

★ ★ ★

Master 25′ Vibratory Finishing Screed, below on the job at San Bernardino, Cal. Note, Screed is attached to box spreader by two cables.

PURPOSE AND RESULTS OBTAINED THEREBY

Here, at last, is a machine so simple that the average laborer with little mechanical knowledge can understand . . . yet get desirable results never obtained before by other methods. The Master Vibratory Finishing Screed is so designed that through the application of vibration on concrete slabs, maximum density, strength and minimum surface variations are obtained. It also allows the placing of concrete with low water content, thereby gaining greater internal strength in the concrete structure.

The Screed in striking off operation rests on pre-set forms or guides and leaves the surface of the slab with corresponding true alignment The Master Screed vibrates uniformly throughout its entire length up to maximum sizes built, assuring uniform compaction and eliminating the element of error due to labor.

The light weight of the Master Vibratory Finishing Screed makes it satisfactory for manual operation on such work as: Highways, concrete floors and other types of slabs; Airport runways, aprons, hangar floors; Warehouse floors; Dock decks; City street paving; Sidewalks; Industrial plant drives; Canal inverts; Bridge decks; Concrete roofs; and many other applications. In addition, the Master Screed is the only highway slab finishing screed that can produce a true parabolic curve crown. All sizes of Master Screeds can be built for any type of crown or invert.

DESIGN: The Master Vibratory Finishing Screed is a simple, sturdy all-steel, vibrating strike-off screed, consisting of a steel Tee-shape and vibrating member spanning the slab, supported at either end by adjustable spring-mounted steel shoes which travel on the forms or guides between which the concrete is deposited.

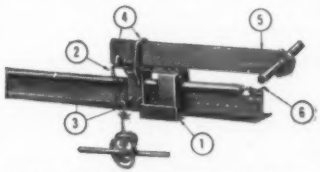
MASTER VIBRATOR FINISHING SCREED (Continued)

Centrally located is a vibrating element securely fastened to the Tee section and driven by an air cooled gasoline engine or electric motor through a Master automatic clutch and Vee belt drive. Vibrations are adjustable from 2000 to 6400 per minute through the use of the engine throttle, thereby providing means of applying the number of vibrations per minute that produces the most satisfactory results in the concrete being used. The spring-mounted end shoes are adjustable for various widths by close drilled holes in the main member (see diagram at right).

The screed is furnished with Draw Ropes at either end for forward or backward movement. Generally on units up to 16 feet long, two laborers are all that are required. Longer screed may require up to four laborers. Weights have been reduced to the minimum, a 20 foot Vibratory Finishing Screed weighing 380 pounds and smaller sizes correspondingly lighter. Movement of the screed from one location to another is quickly made by use of the carrying handles furnished with screed.

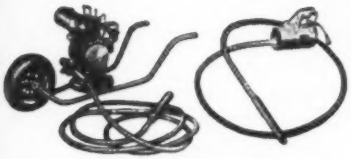
Vibratory screeds to finish slabs of 6 ft., 10 ft., 13 ft., 16 ft., 20 ft., 25 ft., and 26 ft. are standard—longer lengths built to order. All standard screeds adjust down 7' 6" from maximum length, except VS-6 (6' screed), which adjusts down 3' 6". More adjustment can be obtained by drilling vibratory beam in field.

For further details write for Master Bulletin 596

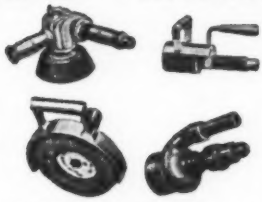


1. Adjustable spring mounted shoes. 2. 2-way draw. 3. Easy adjustment for length. 4. Yoke for lifting. 5. Lifting handle. 6. Remote engine control.

MASTER COST-SAVING EQUIPMENT—FOR IMMEDIATE DELIVERY



Gas or Electric Concrete Vibrators (Catalog No. 689)



Hand Tools for operating from all Master Vibrators and BIG-3 (Catalog No. 683)

BIG-3 for Generation, Tool Operation and Concrete Vibration (Catalog No. 687)

Portable Gas-Electric Generator Plants. Sizes 500 to 17000 Watts (Catalog No. 594)

General Purpose Floodlights

"Power-Blow" Electric Hammer and Spade (Catalog No. 688)

Electric Pavement Breaker (Cat. No. 718)

Gas or Electric Back-Fill Tampers (Catalog No. 699)

Vibratory Concrete Finishing Screed. Sizes 6' to 30' (Catalog No. 596)

"Turn-A-Trowel" for trowelling concrete Sizes 48" or 34" (Catalog No. 685)

Send for illustrated catalog on any item to

MASTER VIBRATOR COMPANY

DAYTON 1, OHIO

"Anti-Hydro," A Durability and Internal Curing Agent for Portland Cement Mixtures

Increases the Strength and Produces Impermeable Results

Construction engineers have specified and insisted on the use of Anti-Hydro in concrete and cement mixtures for more than forty years. Anti-Hydro with any standard brand of portland cement, produces more durable and impermeable concrete of increased early and ultimate strength that otherwise would require special cements and many different single purpose admixtures. These single purpose products often achieve their specific advantages to the detriment of equally desirable properties. Anti-Hydro in concrete and cement mixtures produces all of the following results in one operation and at no added cost for labor:

1. Increased workability.
2. Dispersion of cement.
3. Limited air entrainment without sacrifice of strength.
4. Increased flow with lowered water-cement ratio.
5. Impermeability.
6. Reduced shrinkage.
7. A better bond between new and old concrere and mortars.
8. Uniform curing throughout the entire mass of the concrete of vertical or horizontal construction.
9. Increased early and ultimate strength.
10. Durable concrete without sacrifice of strength.
11. Control time of set of cement.
12. 24 hour use of concrete.
13. Protection of reinforcement.
14. Concrete durable under freeze, thaw and salt action.
15. Hard wearing surfaces.
16. Construction speed and economy.

These results are attested to not only by the commercial recognition accorded Anti-Hydro during the past four decades, but also by tests conducted by government and accredited independent laboratories. These include, in part the laboratories of California Institute of Technology, Case School of Applied Science, Columbia University, E. L. Conwell & Co., Georgia School of Technology, David Kirkaldy & Son, London, England, University of Michigan, Bureau of Standards, H. C. Nutting Co., Raymond G. Osborne, Pittsburgh Testing Laboratories, Smith-Emery Co. and U. S. Engineers.

Anti-Hydro is a liquid integral compound which is added to the gauging water for all portland cement mixtures. It is a calcined solution of colloidal resinates. The formula has remained unchanged since 1904, at which time development of the present formula was perfected by Ferdinand M. Hausling in association with Dr. Thomas B. Stillman.

Since density, impermeability and compressive strength are among the measurable criteria of the effectiveness of curing media, test data are presented demonstrating these and other valuable properties imparted to concrete by gauging portland cement mixtures with "Anti-Hydro."

"Anti-Hydro" increases the plasticity and workability of concrete mixtures. Laboratory results reported by the Case School of Applied Science, Dept. of Engineering Research of the University of Michigan, and California Institute of Technology all confirm the increase of slump produced by "Anti-Hydro" while one U. S. Engineers' Laboratory reports the following comparative results on compressive strength and slump.

A constant W/C ratio of 6.75 gals. per sack was used throughout the test, with Anti-Hydro liquid replacing a like amount of water in the mixes containing this admixture.

Test No.	Without "Anti-Hydro"				With 1½ Ga. "Anti-Hydro" Per Cu. Yd.			
	Slump, Inches	Wt. (Lbs.) Per Cu. Ft.	Strength psi. 7 Days	Strength psi. 28 Days	Slump, Inches	Wt. (Lbs.) Per Cu. Ft.	Strength psi. 7 Days	Strength psi. 28 Days
1	2.0	151.1	2440	3560	2.75	150.1	2650	3780
2	2.0	151.1	2420	3820	3.00	150.6	2670	3700
3	2.25	151.1	2440	3540	3.00	151.7	2560	3640
Avg.	2.08	151.1	2433	3640	2.92	150.8	2627	3707

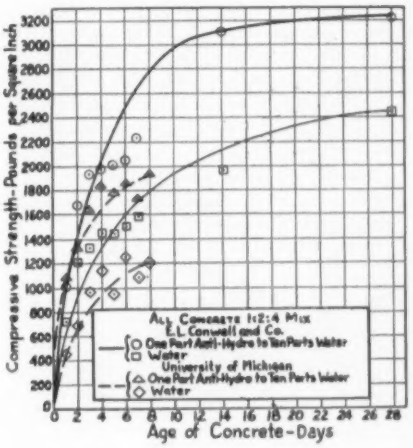
"Anti-Hydro" also increases the tensile strength of portland cement mixtures. This has important bearing upon the bonding qualities of "Anti-Hydro," which were tested by Raymond G. Osborne Laboratories of Los Angeles, California. "This test was made for the purpose of determining the degree of bond possible between a layer of poured concrete to be used as topping and the old concrete floor slab." Smith-Emery Company of Los Angeles reporting on the shear tests state that "the failure in all cases was in the old concrete. There was no apparent failure of the bond." N. B. The bonding material was a thin portland cement grout mixed with "Anti-Hydro" according to specifications.

Increased compressive strength of all ages may be obtained with any standard brand of portland cement through the addition of "Anti-Hydro" to the concrete mix.

The graph shows comparative curves with and without "Anti-Hydro" in the mix as reported by E. L. Conwell & Company, Philadelphia, and Dept. of Engineering Research, University of Michigan, which are representative of innumerable comparative "Anti-Hydro" tests.

"Anti-Hydro" protects reinforcing steel from corrosion. E. L. Conwell & Company, Philadelphia, state in their report of October 9, 1928, "Specimens exposed to the weather up to four weeks showed no sign of corrosion on the steel in mixes containing 'Anti-Hydro.' "

Another function of "Anti-Hydro" is its added protection against frozen concrete and masonry in



winter construction, and the earlier use of the concrete. The H. C. Nutting Company reports on a series of cylinders which, during the initial 24 hours, were kept at 15° F. Of 7 without "Anti-Hydro," 3 were frozen. One (1) containing "Anti-Hydro" out of a total of 7 tested, was found "slightly frozen," but subsequently reached its normal strength.

It is in place to refer to permeability tests on mortars and to permeability and compressive strength tests in which "Anti-Hydro" gauged mixtures were shown to be impermeable while increasing the compressive strength of the concrete, at all ages reported. Many calcium chloride products in these same reports were shown to produce more permeable concrete than the standard untreated concrete of the same mix.

Many admixtures and Special types of cement have been offered from time to time for specific purposes of increased workability, strength, impermeability, dispersion or surface curing. Anti-Hydro has produced impermeable concrete of increased strength and has functioned as a durability and internal curing agent over a long period of years with uniformly satisfactory results.

ANTI-HYDRO WATERPROOFING COMPANY

ANTI-HYDRO
FOR PERMANENT CONCRETE
HARDENS WATERPROOFS

265 Badger Avenue
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Solving CONCRETE Problems

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The Problem... to obtain dense, high-quality concrete
- to prevent formation of cold joints
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The Solution... Plastiment, a Sika product, was successfully used and the above characteristics obtained, plus:
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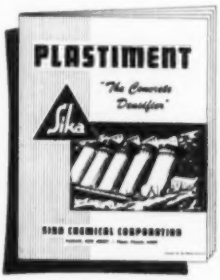

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Sika Products are available for pressure grouting, expansion joints, floor hardening, tile setting, watertight and improved concrete.

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Butler Bin Company 452
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Chain Belt Company of Milwaukee 430-1
Concrete Masonry Products Company 415
Dewey and Almy Chemical Company 450-1
Electric Tamper & Equipment Co. 416-17
Flexible Road Joint Machine Co. 432
Fuller Company 434
Heltzel Steel Form and Iron Co. 454-5
A. C. Horn Company 412
Hunt Process Company 449
Inland Steel Co., The 446-7
Intrusion-Prepakt, Inc., & The Prepakt Concrete Co. 437-40
Jaeger Machine Company, The 418-9
C. S. Johnson Co. 448
Kalman Floor Company 420-1
Koehring Company 424
Lone Star Cement Corporation 428-9
Mall Tool Company 441
Master Builders Company, The 457-464
Master Vibrator Company 466-7
Prepakt Concrete Co., The, and Intrusion-Prepakt Inc. 437-440
Ransome Machinery Co. 435
Raymond Concrete Pile Co. 414
Richmond Screw Anchor Company, Inc. 456
Roberts and Schaefer Company 427
Scientific Concrete Service Corporation 426
Sika Chemical Corporation 470-1
Stearns Manufacturing Co., Inc. 409
United States Rubber Company 433
Vacuum Concrete, Inc. 442-3
Viber Company 422-3
Whiteman Manufacturing Company 445

ACI NEWS LETTER

Vol. 17 No. 4 JOURNAL of the AMERICAN CONCRETE INSTITUTE February 1946

—To provide a comradeship in finding the best ways to do concrete work of all kinds and in spreading that knowledge.

42nd Annual Convention

Monday, Tuesday, Wednesday and Thursday February 18, 19, 20 and 21, 1946

HOTEL STATLER

ADMINISTRATIVE MEETINGS

Advisory Committee, STANTON WALKER, Chairman.
9:30 a. m., Monday, February 18, Grover Cleveland Room

Board of Direction—12:30 (noon) Monday, February 18, Grover Cleveland Room for lunch and continuing afternoon and evening.
(The newly elected Board) 1 p. m. Thursday, Feb. 21, or immediately following adjournment of convention.

TECHNICAL COMMITTEE MEETINGS

(see also Bulletin Board at Registration Desk)

Committee 318, Standard Building Code, A. J. Boase, Chairman, Roy R. Zipprodt, Secretary, 9:30 a. m. Tuesday, February 19, with the expectation of continuing its labors the afternoon of the 21st following the adjournment of the convention and possibly through Friday, the 22nd.

Committee 318, Sub-committee 5, John R. Nichols, Chairman, will meet following first general convention session.

Committee 604, R. W. Spencer, Chairman, will meet—see bulletin board.

Committee 711, Precast Floor Systems for Houses, F. N. Menefee, Chairman, 10:30 a. m. February 19.

CONVENTION REGISTRATION

(All registered non-member visitors are welcome at general sessions—no registration fee)

9.30 A. M. TUESDAY, FEBRUARY 19 TO NOON FEBRUARY 21

Foyer, Niagara Room

in charge of

ETHEL B. WILSON, Assistant Secretary

See Bulletin Board for special notices.

Buy your tickets Tuesday if you wish to attend Luncheon, Wednesday noon
(tickets $2.00, include tips)

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12 noon sharp, LUNCHEON for all announced **program participants** in the sessions of Tuesday afternoon and evening. Presiding officers, Session Leaders, Chairman and Secretary Publications Committee and those who are to present papers or committee reports at the **Tuesday General Sessions of the Convention.**

2.00 P. M. TUESDAY, FEBRUARY 19

President DOUGLAS E. PARSONS, Chairman

Appointment of Tellers, to canvass ballots of the Annual Election.

"Proposed Minimum Standard Requirements for Precast Concrete Floor Units"

Report of Committee 711, F. N. MENEFEE, Charman

Presented by reference (ACI Jl. January, 1946) on a motion for adoption as an ACI Standard

NEW DEVELOPMENTS IN CONCRETE DESIGN METHODS

A. J. BOASE, Session Leader

Manager Structural Bureau, Portland Cement Association

"Radiant Heating by Reinforced Concrete"

JOHN R. NICHOLS

Consulting Engineer, Boston

There is evidence, Mr. Nichols believes, that radiant heating pipes in structural concrete will not be as destructive as an atomic bomb. Since there is great value and convenience in this heating method, he believes the prohibiting codes must be scrutinized to make sure that progress is not needlessly impeded.

"The Lattice Analogy in Concrete Design"

DOUGLAS McHENRY,
U. S. Bureau of Reclamation

The author will show that many complicated problems in stress distribution can be solved by a method which involves only simple arithmetic. The field of application of the lattice analogy is comparable to that of the photoelastic polariscope, but the elaborate equipment of photoelasticity is replaced by an ordinary computing machine.

"The Use of Prestressed Concrete in Floating and Land Structures"

HENRI MARCUS,
Bureau of Yards and Docks, Navy Department

This paper describes the design and construction of a concrete barge in which prestressed reinforcing steel was used. It includes a brief discussion of laboratory tests of prestressing which preceded the design.

"Hipped Plate Construction"

GEORGE WINTER,
Assistant Professor of Civil Engineering, Cornell University

The author will describe a method of design which utilizes effectively the monolithic continuous character of concrete. This type of construction, known abroad as "Faltwerke," has been little used in this country. Professor Winter has had occasion to design and see several substantial structures of this type erected during a number of years spent as a consultant in Russia.

8.00 P. M. TUESDAY, FEBRUARY 19

Vice President, HARRISON F. GONNERMAN,
Chairman

ENTRAINED AIR IN CONCRETE

F. H. JACKSON, Session Leader
Principal Engineer of Tests, Public Roads Administration

"Laboratory Studies of Concrete Containing Air-Entraining Admixtures"

By C. E. WUERPEL,
Engineer in Charge, Central Concrete Laboratory, North Atlantic Division, U. S. Corps of Engineers

Mr. Wuerpel's lead contribution to the symposium will epitomize the high points of two recent papers published in the ACI Jl.: in September, 1945, field experiences of the War Department; in the current February issue the results of a considerable laboratory investigation (both papers available in separate prints; copies available at registration desk for those who wish to participate in discussion).

TEN—10-MINUTE CONTRIBUTIONS TO THE SESSION THEME

"INDIANA EXPERIENCES IN HIGHWAY CONSTRUCTION"
S. W. BENHAM, Indiana Highway Commission.

"RESULTS OF RECENT LABORATORY STUDIES"
STANTON WALKER, Director of Engineering, National Sand and Gravel Association.

"FIELD EXPERIENCES IN THE NORTHEASTERN STATES"
LEE ANDREWS, Portland Cement Association.

"A MECHANICAL DISPENSING DEVICE FOR AIR-ENTRAINING AGENT"
E. M. BRICKETT, Dewey and Almy Chemical Co.

"A DEVICE FOR MEASURING ENTRAINED AIR IN CONCRETE"
W. H. KLEIN, Vice-President and General Manager, Pennsylvania-Dixie Cement Corp.

"ENTRAINED AIR—A FACTOR IN THE DESIGN OF CONCRETE MIXES"
W. A. CORDON, Engineer, U. S. Bureau of Reclamation.

"EXPERIENCES IN READY MIX OPERATIONS"
ALEXANDER FOSTER, Jr., Warner Company.

"EXPERIENCES IN PENNSYLVANIA HIGHWAYS"
W. H. HERMAN, Chief Research Engineer, Pennsylvania Department of Highways.

"TESTS WITH BLENDS OF NATURAL AND PORTLAND CEMENTS"
W. F. KELLERMANN, Senior Materials Engineer, Public Roads Administration.

"RECENT TEST RESULTS"
HENRY L. KENNEDY, Manager Cement Division, Dewey and Almy Chemical Co.

8 A. M. WEDNESDAY, FEBRUARY 20

8 A. M. BREAKFAST for announced program participants in the session of Wednesday afternoon: Presiding officers, Session Leaders, Chairman and Secretary Publications Committee and those who are to present papers or Committee reports at the general session Wednesday afternoon.

9.30 A. M. WEDNESDAY, FEBRUARY 20

CONCRETE RESEARCH

An Open Session of ACI Committee 115, Research

MORTON O. WITHEY, Chairman; FRANK E. RICHART, Vice-Chairman, S. J. CHAMBERLIN, Secretary

Past-President FRANK E. RICHART, Presiding

This annual session has grown in importance since its inauguration at the Institute's 33rd Annual Convention in New York in 1937. As in former years there is no stenotypist to record the series of brief papers or informal reports on concrete technics and initial results in projects under way. Most of those contributing are doing so with the understanding there will be

no published record except as specific release is made to the Institute for its subsequent publication. Some laboratories do not want publicity until investigations are complete for a fully considered record. With this understanding Members and non-members attending the convention will be welcome at this open committee session. The only way to be sure of what the session develops is to be there.

12 O'CLOCK NOON, WEDNESDAY, FEBRUARY 20, ACI LUNCHEON

This luncheon is for ACI Members and friends who bought their tickets Tuesday or early enough Wednesday morning to "get their noses in the pot" as the old saying goes. The price is $2.00 and that includes the "tip." Gather around the board with your old or new friends and save time in these restaurant-crowded days.

2 P. M. WEDNESDAY, FEBRUARY 20

MAINTENANCE AND REPAIR OF CONCRETE

Vice-President STANTON WALKER, Chairman

RODERICK B. YOUNG, Session Leader
Hydro-Electric Power Commission of Ontario

presenting an introductory paper on the general theme

"Two Special Methods of Restoring and Strengthening Concrete Structures"

J. W. KELLY
Associate Professor of Civil Engineering, University of California

and B. D. KEATTS
Engineer, Intrusion-Prepakt Inc.

See ACI JOURNAL, February, 1946 (or separate print) where Messrs. Kelly and Keatts descrive methods of repairing masonry structures, by which the old deteriorated mortaris washed from the masonry, in successive steps, and replaced with firm grout. Other restorations are made by consolidating aggregate, placed in patched areas of defective concrete, with grout forced into the voids of the aggregate.

"Maintenance of Heavy Concrete Structures"

CLAY C. BOSWELL and ALBERT C. GIESECKE
Minnesota Power & Light Co.

See ACI JOURNAL, February 1946 (or separate print) for the paper describing repair methods on a dam and contrasting the construction methods with those which have made repair unnecessary on a similar but older structure.

"Maintenance and Repair of Portland Cement Concrete Pavements"

A. A. ANDERSON

Manager Highways and Municipal Bureau, Portland Cement Association

Maintenance and repair methods and the equipment used are described and well illustrated.

"Repair of Concrete Chimneys with Minimum Boiler Operation Interference"

W. M. BASSETT

Engineer of Structures, New England Power Service Co.

and MILES N. CLAIR

Vice-President, The Thompson & Lichtner Co., Inc.

The title carries an important consideration of the job reported—the effort to avoid, so far as possible, stoppage of operations while repair was under way.

"Hydraulic Structure Maintenance Using Pneumatically Placed Mortar"

W. L. CHADWICK

Chief Civil Engineer, Southern California Edison Co.

Mr. Chadwick's contribution features methods which are the subject of study by ACI Committee 805, assigned to the preparation of practice recommendations for the application of mortar by pneumatic pressure—a committee of which the author is chairman. The paper, giving the experiences of the Southern California Edison Company, which maintains concrete structures high in the Sierra Mountains, will be presented by R. W. Spencer, Civil Engineer of the company.

"Maintenance and Repair of Concrete Bridges on the Oregon Highway System"

G. S. PAXSON

Bridge Engineer, Oregon State Highway Department

See ACI Journal, November, 1945 (or separate print) where this illustrated paper was published. It describes restoration procedures, followed by waterproofing treatments which have effectively stopped progressive deterioration of the concrete. This paper will be presented by R. F. Blanks, U. S. Bureau of Reclamation.

DINNER 6.15 P. M. WEDNESDAY, FEBRUARY 20

DINNER for announced participants in the sessions scheduled for Wednesday evening and Thursday morning: Presiding Officers, Session Leaders, Chairman and Secretary Publications Committee and those whose names appear as contributors to these two sessions.

8.00 P. M. WEDNESDAY, FEBRUARY 20

President DOUGLAS E. PARSONS, Chairman

Report of Tellers on the Annual Election of Officers and other members of the Board of Direction and of the members of the 1946 Nominating Committee.

Presentation of Awards by President Parsons:

LEONARD C. WASON, Medal for "Noteworthy Research" to

BARTLETT G. LONG, HENRY J. KURTZ and THOMAS E. SANDENAW

for the work reported in their 1945 paper:
(January JOURNAL)

"An Instrument and a Technic for Field Determination of the Modulus of Elasticity and Flexural Strength of Concrete (Pavements)"

LEONARD C. WASON Medal for the "Most Meritorious Paper" of ACI Proceedings Volume 41 (February, 1945, JOURNAL) to

CLARENCE RAWHOUSER
for his paper

"Cracking and Temperature Control of Mass Concrete"

American Concrete Institute Construction Practice Award for "a paper of outstanding merit" on concrete construction practice to

LEWIS H. TUTHILL

for his contribution to the January 1945 JOURNAL of the American Concrete Institute:

"Concrete Operations in the Concrete Ship Program"

The HENRY C. TURNER Medal
"for notable achievement or service" to

JOHN LUCIAN SAVAGE

"in recognition of long and distinguished service in the design of hydraulic structures, including some of the world's most notable dams."

ROBERT F. BLANKS, Chairman Publications Committee, Presiding

Address by the Retiring President

DOUGLAS E. PARSONS,

Chief, Division of Clay and Silicate Products,
National Bureau of Standards

"Curing Concrete with Sealing Compounds"

ROBERT F. BLANKS, H. S. MEISSNER and LEWIS H. TUTHILL

United States Bureau of Reclamation

This paper to be presented by Mr. Meissner supplies data evaluating concrete curing by membrane treatment in terms of equivalent moist curing. It discusses the preferred methods of curing by use of sealing compounds and outlines a specification and acceptance test for their purchase.

"Behavior of Concrete Structures under Atomic Bombing"

CAPTAIN E. H. PRAEGER (CEC), USNR

Captain Praeger, formerly design manager of the Bureau of Yards and Docks, has recently returned from Japan, where he inspected the results of atomic bombing. His paper will be illustrated by steropticons and the ACI audience will have an opportunity to learn what the effects are of the world's greatest weapon of destruction.

"Should Portland Cement Be Dispersed?"

T. C. POWERS

Manager of Basic Research, Portland Cement Research Laboratory

a brief epitomization of a paper published in The ACI JOURNAL for November 1945—also available in separate prints.

9.30 A. M. THURSDAY, FEBRUARY 21

ROBERT F. BLANKS, Chairman

NEW DEVELOPMENTS AND WAR EXPERIENCES IN CONCRETE CONSTRUCTION METHODS

LEWIS H. TUTHILL, Session Leader

"Proposed Recommended Practice for the Construction of Concrete Farm Silos"

Report of Committee 714, WILLIAM W. GURNEY, Chairman

Presented by reference (see ACI JOURNAL Jan. 1944 and revisions Jan. 1946) on a motion for adoption as an ACI Standard.

"Using Coral Aggregates at the Advanced Bases of the Navy"

Commander I. S. RASMUSSEN (CEC) USNR

Commander Rasmussen has been on many projects where coral aggregates have been used in the construction of advanced bases. The Navy has made best use of the materials at hand as our armed forces fought the hard road back and Commander Rasmussen will tell us about it.

"Precast Concrete Structures"

A. AMIRIKIAN,

Principal Engineer, Bureau of Yards and Docks, U. S. Navy

This paper will deal with the application and principles of precasting to such specific structures as storehouses, barges, floating caisson gates, drydocks and hangars.

"Thin Wall Concrete Ship Construction"

F. R. MacLEAY,

Chief Engineer, Corbetta Construction Co.

As a sequel to Mr. Amirikian's paper the author presents the developments of construction methods for pouring thin concrete walls. These methods (illustrated) were successfully used in the construction of concrete leading craft.

"H-Beam and Prepakt Method of Pier Construction"

C. P. DISNEY,

Bridge Engineer, Canadian National Railways

Mr. Disney will discuss a projected plan of rebuilding the Quebec Bridge and reconstruction of many bridges, using a novel method for building piers around central H-beam piles.

"Harnessing the Atmosphere for Concrete Construction"

V. S. MURRAY,

Bridge Engineer, Ontario Department of Highways

With film, this author will illustrate how vacuum methods are used to precast and erect light-weight precast structural units.

"Observations on a Post-War European Journey"

MYRON A. SWAYZE,

Director of Research, Lone Star Cement Corp.

Mr. Swayze was in Germany last summer and members of ACI will enjoy hearing this informal report by one who would note the things that other engineers would like to hear about.

— Adjournment —

ACI Awards Announced

Acting on the report of its Medals Award Committee, the Board of Direction announces:

To John Lucian Savage, a member of the staff of the United States Bureau of Reclamation for 34 years and since 1924 it's Chief Designing Engineer:* the Henry C. Turner Medal (founded 1927 by Past President Turner) "to be awarded annually, but not more often, for notable achievement in or service to" the field of concrete.† The medal is of gold and is accompanied by a certificate of award bearing, in the present instance, this citation:

> *"in recognition of long and distinguished service in the design of hydraulic structures including some of the world's most notable dams."*

To Bartlett G. Long, Henry J. Kurtz and Thomas E. Sandenaw,‡ the Leonard C. Wason Medal for noteworthy research as reported in their Institute paper:

> *"An Instrument and a Technic for Field Determination of the Modulus of Elasticity and Flexural Strength of Concrete (Pavements)."*

published ACI JOURNAL January 1945. The bronze medal (in triplicate) is accompanied in its presentation to each of the authors, by a certificate of the award.

To Clarence Rawhouser, of the Bureau of Reclamation, the Leonard C. Wason medal for each year's "most meritorious paper" on the basis of his

> *"Cracking and Temperature Control of Mass Concrete"*

published ACI Journal, February 1945. The bronze medal is in its presentation accompanied by a certificate of the award.*

The Wason medals were founded by Past President Wason (the Institute's second president, 1915-1916) in 1917.

To Lewis H. Tuthill, of the Bureau of Reclamation* the first American Concrete Institute Construction Practice Award, founded 1944 for "a paper of outstanding merit on concrete construction practice": This is based on Mr. Tuthill's paper:

> *"Concrete Operations in the Concrete Ship Program"*

Mr. Tuthill will receive with his certificate of award U. S. Series E Bonds of a maturity value of $300.00.

Presentation of these awards will be from the hands of President Parsons, the evening of February 20, at the Institute's 42nd Annual Convention (see program in these pages).

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*See brief biographical sketch in pages which follow.

†Previous Turner Medalists: Arthur N. Talbot, 1928, William K. Hatt, 1929; Frederick E. Turneaure, 1930; Duff A. Abrams, 1932; John J. Earley, 1934; Phaon H. Bates, 1939; Ben Moreell, 1943.

‡For biographical notes on the authors see following pages.

The Medalists

John Lucian Savage—Turner Medal

John L. Savage was born on a Wisconsin farm near Cooksville, Dec. 25, 1879. In 1903 he received a B. S. degree in Civil Engineering at the University of Wisconsin. For 34 years he has been a member of the Bureau of Reclamation staff, and since 1924 has been its Chief Designing Engineer. His service with the Bureau began on the Minidoka Project in Idaho after graduation from school. For four years he worked in the Boise office designing irrigation structures under the guidance of Arthur P. Davis, Chief Engineer, and A. J. Wiley, Consulting Engineer. In 1908 Mr. Savage left the Reclamation Service to become associated with Mr. Wiley in an engineering practice in Boise. He has been an ACI member since 1928.

During this time, he designed many important structural works including the Salmon River Dam, the Swan Falls Power Plant on the Snake River, the Barber Power plant on the Boise River, the Oakley Reservoir Dam, and the American Falls Power Plant. He also received a special assignment from the Reclamation Service to design gates for the Arrowrock Dam on the Boise River.

In 1916 Mr. Savage re-entered the Reclamation Service in charge of civil engineering designs in the newly organized office of the Chief Engineer at Denver, Col., where important design work of the Bureau is done. In 1924 he was placed in charge of all electrical and mechanical designing with the title of Chief Designing Engineer, the position which he held until his retirement, April 30, 1945.

Mr. Savage was the designing engineer on the three largest concrete dams in the world—Grand Coulee, Shasta, and Boulder. Other famous dams that he has designed include the Norris Dam in the Tennessee Valley, and the Wheeler, another of the key structures in the basis-wide development of that area; Friant Dam, important co-worker of Shasta Dam in the Central Valley Reclamation Project (California); and Madden Dam at the Panama Canal. The Marshall Ford Dam in Texas, Imperial Dam on the Arizona-California border, Parker Dam on the Colorado River, and American Falls Dam (Idaho) are among the numerous Reclamation dams he has designed.

At the request of the Chinese government, Mr. Savage in 1944 made extensive surveys for the postwar development of China's great natural resources, including the huge Yangtze Gorge Project to irrigate 10,000,000 acres of land and develop some 10,000,000 kilowatts of electric power capacity. He also spent several months in India, visiting projects for irrigation and other developments in the United Provinces, Bengal, Madras, Mysore, and the Punjab.

When a cable came from London in 1940 asking the United States for Mr. Savage's assistance in connection with the Burrinjuck Dam in Australia, attention was directed to the law which prohibits officials of the United States from accepting emoluments from foreign countries. Rather than delay the voyage in this emergency Mr. Savage wired former Commissioner John C. Page of the Bureau of Reclamation: "Any assistance given Government

John Lucian Savage

of New South Wales will be gratis and I shall not accept any fee or other form of compensation or any reimbursement." He said he needed a vacation, anyway.

In 1940, Congress passed a bill specifically authorizing the President to send Mr. Savage to Australia and India as an engineering consultant. He spent four months in Australia in 1941-42 as consultant on two projects—the Upper Yarra Dam for the Melbourne and Metropolitan Water Board of Works and the Kiewa Dam for the State Electricity Commission of Victoria.

Engineer Savage has been honored many times for his achievements. On his recent visit to China he received a gold medal from the Ministry of Economic Affairs for Humanitarian Services. The American Society of Civil Engineers last year awarded him the John Fritz medal, one of the highest honors of the Engineering profession. In 1934, the University of Wisconsin conferred upon him the honorary degree of Doctor of Science. He was presented in 1937 with one of the most prized awards given to anyone in the engineering profession—the Gold Medal for Outstanding Engineering Service of the Colorado Engineering Council, which represents all engineering organizations in the State. The award had been made but once in the previous ten years.

"Jack Savage in a very real sense has epitomized the Bureau of Reclamation. He grew up with the organization. His extraordinary gift is the ability to bring a fresh mind to new problems in engineering. His approach is daring, but his plans are sound. Neither the necessity of building the highest dam in the world in Boulder Canyon nor the requirement for the construction of the most massive concrete dam so far conceived at Grand Coulee on the Columbia River dismayed Jack Savage. He simply proceeded to design the dams, and they now are the pride of the engineering world."—Comments of Commissioner of Reclamation Harry W. Bashore on Mr. Savage's decision to retire at 65.

Mr. Savage has now been appointed Consulting Engineer for the Bureau of Reclamation, and is serving as Consulting Engineer for several foreign governments. One of the important works on which he will advise is the Yangtze Gorge Dam near Ichang, China, which structure will be larger than Grand Coulee.

Lewis Hamilton Tuthill—ACI Construction Practice Award

Lewis Tuthill was born in Philadelphia, November 24, 1899. He was graduated from Oregon State College with B. S. degree in Engineering in 1920. Since graduation, he has been continuously engaged in design and construction work on irrigation, water supply projects and the building of dams, with the technique and control of concrete as his primary and absorbing interest. His first employment was with the Grants Pass Irrigation District, in Oregon; next the Merced Irrigation District in California, where he was asso-

ciated with the control of concrete on Exchequer Dam; and, following this, the same detail on Malones Dam for the Oakdale and South San Joaquin Irrigation District. After a year of private consulting practice, concrete control work on Pardie and Hogan Dams for the East Bay Municipal District and City of Stockton, California, next occupied his attention. During seven years with the Metropolitan Water District of Southern California, he prepared specifications for 4½ million yards of concrete work and materials, passed upon the acceptance of cement and various materials for the immense aqueduct from the Colorado River to the Los Angeles area, and prepared a manual of inspection. Since 1939 he has been a member of the staff of the Bureau of Reclamation, still pursuing the profession of concrete engineer, and assisting in editing the Bureau's Concrete Manual. During a leave of absence from the Bureau, he spent two of the war years supervising concrete quality on the U. S. Maritime Commission's concrete ship construction program. For a description of this work, he now receives the first A.C.I. Construction Practice award based on its record of good practice applicable to many other types of structure than ships.

Mr. Tuthill was Chairman of Committee 614, whose report on "Recommended Practice for Measuring, Mixing and Placing Concrete" became an ACI Standard in 1942. The ACI Honor-Roll records attest the fact that he has been a very active worker for new ACI members. His contributions to and wide knowledge of concrete construction practice led to his recent appointment to the Advisory committee, as Chairman of Department 600, Construction. He has been an ACI member since 1926.

Clarence Rawhouser—Wason Medalist

Clarence Rawhouser was born York, Pa., February 1, 1902. He entered the University of Cincinnati in 1921, and enrolled in the cooperative course in Civil Engineering. After an interruption by illness, of his formal training, he returned to the University and received the CE degree in 1930. His cooperative educational training extended even farther back than college days; through alternate work and school periods he completed apprenticeship in wood patternmaking with the York Ice Machinery Corp. while attending the Industrial Course in York High School.

In 1929 he received the Cincinnati Engineers' Club annual award to the outstanding senior of the U. of C. College of Engineering. He was employed by the U. S. Engineers Office of the War Department in Cincinnati for a year, where he was engaged in studies of flood control of the rivers of the Cincinnati District. He has been in the dam engineering division of the Bureau of Reclamation in Denver, since 1930. Since that time he has been directly concerned with the problems of temperature control, artificial cooling, and observation of structural behavior of all concrete dams constructed by the Bureau, beginning with the pioneering work of that nature on Boulder Dam. He has contributed materially to the plans and methods used for controlling the temperature of Seminole, Bartlett, Parker, Grand Coulee, Marshall Ford, Shasta, Friant, and Altus dams. For suggestions made for special temperature-control measures at Friant Dam he received a Department of Interior Award of Excellence. He has been an ACI member since 1937.

He obtained the MS degree in Civil Engineering from the University of Colorado in 1943. He is an Associate Member of the American Society of Civil Engineers and a member of Tau Beta Pi.

Bartlett G. Long, Henry J. Kurtz and Thomas Sandenaw—Wason Medalists

The three collaborating authors are all ACI members—Long since 1934, Kurtz since 1942, Sandenaw since 1943, all of them until recently actively identified with the Army Engineer Corps' Cin-

cinnati Testing Laboratory, at Mariemont, Ohio.

"Bart" Long, well known to many ACI members, severed his connection with the laboratory in 1944 and he and Mrs. Long have since been at their ranch at Pecos, N. M.

Mr. Long was born in St. Louis, Missouri, Apr. 3, 1892. He attended the University of Missouri, from Sept. 1911 to June, 1912, and the Washington University, from Sept. 1912 to June, 1914. He was a first lieutenant in World War I from July 26, 1917 to May 2, 1919, having served as an instructor in the Air Service.

His professional experience includes employment by Nelson Cunliff, Engineer, and Park Commission, St. Louis, Missouri; Florida State Road Dept., Tallahassee; Woods-Hoskins-Young Co., Chicago; State Highway Department, Santa Fe, New Mexico; Bureau of Reclamation, Denver, Colo.; War Department, U. S. Engineers, Seattle; U. S. National Park Service, San Francisco; War Department, U. S. Engineers, St. Louis, and Mariemont, Ohio.

Mr. Kurtz was born in Flint, Mich., Jan. 8, 1897; was graduated from Michigan State College, B.S. in 1921, M.S. in 1928.

His professional experience includes employment by Consumers Power Co., Battle Creek, Mich.; Commonwealth Power Corp., Jackson, Mich.; Pacific Telephone & Telegraph Co., Sacramento, Calif.; Bell Telephone Laboratories, New York City; Remler Radio Co., Inc., San Francisco; National Bureau of Standards, Washington, D. C.; American Society for Testing Materials, Washington, D. C.; U. S. Coast Guard, Cambridge, Mass.; War Department, U. S. Engineers, Mariemont, Ohio.

Mr. Sandenaw took his B. S. in industrial chemistry from Montana State College in 1934; attended Ohio State University for graduate work in chemistry in 1935-36; was employed in the chemistry section United States Engineers laboratory Fort Peck, Montana; 1936-38, took an M. S. degree in chemistry from Carnegie Institute of Technology in 1939; analytical chemist in the U. S. Navy Inspection Laboratory, Marshall, Penna., 1939-41; employed in the chemical section of General Materials Laboratory, special engineering division, the Panama Canal, Diablo Heights, C. Z. 1941-43; in the concrete section Cincinnati Testing Laboratory, Mariemont, Ohio, 1943-44. From there he went to the RCA Victor Division of Radio Corp. of America as a chemist in the engineering department of the vacuum tube manufacturing plant.

WHO'S WHO

Clay C. Boswell and Albert C. Giesecke

collaborate in a paper describing repair methods on a dam (p. 377)—part of a 1946 convention session on concrete maintenance and repair.

Mr. Boswell, an ACI Member since 1934, is Vice President and Chief Engineer of Minnesota Power & Light Co. Graduating from Missouri University in electrical engineering in 1915, Mr. Boswell has been identified with various public utility operations almost continuously since that time. He served his country during World War I, spending almost a year in France in command of an anti-aircraft searchlight detachment.

Mr. Boswell is a Member, American Institute of Electrical Engineers, and active in many other professional and civic organizations.

Mr. Giesecke, has served as Hydraulic Engineer for Minnesota Power & Light Co. Duluth, Minnesota since 1923. Prior to that time he was identified with important work in Latin America; also with Utah Power & Light Co., Pacific Power & Light Co., and several irrigation projects in Utah, Washington and California. Mr. Giesecke served overseas in a machine gun combat unit during World War I,—also on General Staff assignments. His ability to speak both German and Spanish has

been of substantial advantage to him in the practice of his profession. He is an Associate Member of American Society of Civil Engineers.

J. W. Kelly and B. D. Keatts

collaborated in the paper (p. 289), which is one of a series to be presented at the ACI Convention this month, on maintenance and repair of concrete structures.

Professor Kelly, who has much work to his credit in his 20 years of ACI Membership (see p. 69 ACI 1945 Directory) needs no introduction to his present audience.

Mr. Keatts has been an ACI member since 1942, was graduated from the University of Illinois in 1924 with the degree of B. S. in General Civil Engineering. His first work after graduating was in the planning and specification's office of the Illinois State-Highway Department at Springfield for one year, and then with the Bridge Department of the Missouri Pacific Railroad at St. Louis where a small part of his time was in preparing plans for concrete structures; most of it as Resident Engineer on bridge construction under contract. He also had charge of maintaining the railroad's two river ferries for freight train service at St. Louis, Missouri and at Natchez, Mississippi. After four and a half years with Missouri Pacific Railroad he went to Stone and Webster Engineering Corp. at Chicago for four years on the construction of equipment which was manufactured in the Chicago territory for installation in dams and industrial plants.

In the depression, he found haven with other engineers at the Century of Progress world's fair at Chicago, three years during the construction period, two years of operation, and part of the demolition period.

Next he was with the Portland Cement Association in the States of Illinois and Wisconsin—his engineering work with PCA on contacts and instruction with engineers, technicians, and builders in the construction field.

He has been with Intrusion-Prepakt, Inc. for six years in the field locating proper sources of materials and getting them to the various jobs, establishing new field, laboratory-developed methods, and working with customers' engineers on special reconstruction problems. On January 1, 1946 he was given charge of the company's activities in the Eastern States.

Charles E. Wuerpel

is already well known to the ACI audience. His paper (p. 305) his second recent contribution on air-entrainment—field experience (September 1945) and now laboratory results. He will draw from both for a brief epitomization to lead the convention session on that subject.

Gerald Pickett

appeared in the January Journal with part 1 of "Shrinkage Stresses in Concrete;" Part 2 starts page 361.

Honor Roll

February 1, 1945 to January 24, 1946

Rene Pulido y Morales, in Havana, Cuba heads the list with 24 new Members proposed since Feb. 1, and only a week to go to end the Honor Roll year.

Rene Pulido y Morales	24
Roy Zipprodt	10
H. F. Gonnerman	7
Harry B. Dickens	5
A. Amirikian	3
J. A. Croft	3
Ernst Gruenwald	3
Dean Peabody Jr.	3
J. H. Spilkin	3
Charles S. Whitney	3
Francis MacLeay	2½
D. E. Parsons	2½
Charles E. Wuerpel	2½
J. B. Alexander	2
C. Blaschitz	2
H. W. Cormack	2
J. W. Kelly	2
Calvin C. Oleson	2
O. G. Patch	2
C. H. Scholer	2
Stanton Walker	2
J. M. Wells	2

J. C. Witt	2
Ben E. Nutter	1½
F. E. Richart	1½
O. A. Aisher	1
Michel Bakhoun	1
H. C. Bruce	1
H. Victor Carman	1
W. Fisher Cassie	1
A. R. Collins	1
R. F. Dierking	1
H. F. Faulkner	1
P. J. Freeman	1
B. F. Friberg	1
J. K. Gannet	1
Arturo Gantes	1
Stanley S. Haendel	1
W. S. Hanna	1
G. H. Hodgson	1
F. B. Hornibrook	1
V. P. Jensen	1
L. I. Johnstone	1
William G. McFarland	1
Denis Matthews	1
Charles E. Morgan	1
H. W. Mundt	1
Y. G. Patel	1
J. R. Pattilo	1
A. F. Penny	1
Kenneth Powers	1
Guy Richards	1
A. T. Rogers	1
Simeon Ross	1
John A. Ruhling	1
Herman Schorer	1
Byram Steel	1
G. W. Stokes	1
H. D. Sullivan	1
H. W. Sullivan	1
Wm. Summers Jr.	1
M. A. Timlin	1
J. W. Tinkler	1
Lewis H. Tuthill	1
Maxwell Upson	1

The following credits are, in each instance "50-50" with another Member—"half a member" to each name listed.

Birger Arneberg	J. H. Chubb
E. E. Bauer	Miles N. Clair
E. W. Bauman	Arthur P. Clark
P. G. Bowie	R. R. Coghlan
C. H. Chubb	R. B. Crepps
R. A. Crysler	Raymond Osborne
Harmer E. Davis	Lucien Perrault
J. L. Drueke	Milos Polivka
P. M. Ferguson	Jerome Raphael
Alexander Foster	Chas. S. Rippon
G. L. Freeman	D. F. Roberts
Grayson Gill	D. O. Robinson
E. A. Gramstorff	Kanwar Sain
N. M. Hadley	J. L. Savage
W. C. Hanna	Oskar Schreier
Carl W. Hunt	A. L. Strong
A. Dovali Jaime	A. C. Trice
W. R. Johnson	K. Tautsumi
Paul A. Jones	J. H. de W. Waller
H. J. McGillivray	S. J. Warberg
R. E. McLaughlin	A. R. Waters
Adolph Meyer	David Watstein
A. F. Moore	H. J. Whitten
O. F. Moore	George Winter
E. Nennigar	Harry C. Witter
M. D. Olver	S. H. Woodward
C. E. O'Rourke	K. B. Woods

L. Zeebaert

New Members

The Board of Direction approved 22 applications for Membership (18 Individual, 3 Junior, 1 Student) received in December as follows:

Charters, C. M., c/o British American Oil Co. Ltd., 703 Royal Bank Bldg., Toronto, Ont., Canada

Dixon, Frederick R., 43 Fairlight St., Five Dack, Sydney N. S. W., Australia

Englander, Harry, 58 Campbell Ave., Williston Park, New York N. Y.

Ferulano, Paolo Emilio, Lista de Correo, Caracas-R de Vene uela, S. A.

Frank, Jacob, c/o R. S. Noonan Inc., 11 East Market St., York, Pa.

Gondolfi, D. E., 707 W. Main St., Ottawa, Ill.

Haller, Karl H., 824 Claremac Dr., West Palm Beach, Fla.

Jameson, R. O., 5519 McComas Ave., Dallas 6, Texas

Johnston, W. R., 1403-W. 45th Street, Seattle, Wash.

Kauer, T. J., Wire Reinforcement Institute, 1049 National Press Bldg., Washington 4, D. C.

Kilkenny, Paul E., 2575 Le Conte Ave., Berkeley 4, Calif.

Kinne, Raymond C., 329 Fountain Ave., Dayton, Ohio

Koo, Hia chang Benjamin, 214 Linden Ave., Ithaca, N. Y.

Martinez, Milton R., 301 Bryant Ave., Ithaca, New York

Mauldin, W. O., 412 R. A. Long Bldg., 928 Grand Ave., Kansas City 6, Mo.

Parme, Alfred L., 5234 S. Dorchester Ave., Chicago 15, Ill.

Pichel, Ernest, 105 W. 72nd St., New York 23, N. Y.

Plum, Niels M., 17 ved dammen, Bagsevaerd, Denmark

Quilty, Thomas Patrick, 884 Riverside Dr., New York 32, N. Y.

Shotwell, J. G., Box 888, Albuquerque, New Mexico

Tamanini, Flory J., 191 Yale Drive, Alexandria, Va.

Wood, Reginald Escott, Narriott & Wyhna Sts., Holland Park, Brisbane S. E. 3, Queensland, Australia

Fred F. Van Atta

Fred F. Van Atta Joins ACI Staff

Erstwhile Major, now Mr. Fred F. Van Atta joined the Institute staff January 16, three weeks home from the Philippines, where as an army engineer, he applied himself to bridge reconstruction near Manila. Previously he had spent four and a half years with T. V. A. (Kentucky Dam) and later as a reserve officer in the Army was identified as assistant to The District Engineer, Charleston, S. C. with various projects in the Charleston area.

Since January 16 he has been doing research work—finding out about ACI, its objects, policies, practices, how it works, its organization and the varied details of producing Journals and other ACI publications. He is finding out what makes ACI tick; has learned something of why it doesn't always *click* on time. To *that* he will shortly begin to put his weight (and for that ACI isn't counting on mere avoirdupois). He will attend the convention in Buffalo this month to get acquainted; to see ACI in action.

Here are further details of his record:

Mr. Van Atta was born in Detroit, Mich., July 26, 1912; was graduated from Michigan State College, 1934 with a degree of B. S. in C. E. "with high honor". That summer he worked as technical assistant in research work on aggregate in the concrete laboratory of Michigan Engineering Experiment Station, Michigan State College. Next he was with the Coast and Geodetic Survey for ten months in five western states on precise level surveys; then Tennessee Valley Authority, Gilbertsville, Ky., September 1935 to January 1940, on foundation investigations and the early stages of construction of Kentucky Dam. Next, to vary his experience, January 1940 to March 1941 with U. S. Engineer Dept., Los Angeles, Calif. as Junior Civil Engineer (Hydraulic) in the development of hydrology and hydraulic design, followed by related work in hydraulic model studies.

Just before the war broke, he was with Jarvis Engineering Works, Lansing, Mich. for about five months on structural steel design and field work.

He entered the army Oct. 13, 1941 from the Officers Reserve Corps. As assistant to the District Engineer, Charleston, S. C., he was in charge of the preparation of plans and the construction of Aircraft Warning stations, concurrently assisting in the preparation of plans and specifications on fortifications for the Harbor Defenses of Charleston. Later he was Assistant Chief of the Military Engineering Section for Construction of airports and cantonments, and still later Assistant Chief of the Operations section of the District.

In June 1942 he was transferred to the North Charleston Area Office as chief of the Engineering Section in charge of engineering design and the preparation of plans and specifications for railroad facilities, concrete and timber wharves, earth fill, concrete and asphalt paving, roads, hospitals and cantonments at an army airbase, port of embarkation and general hospital.

War Department orders transferred him to Ft. Belvoir, Va., July 1, 1944 for ten weeks of training. Then followed short tours of duty at Fort Leonard Wood, Camp Gordon Johnston and Ft. Belvoir, prior to going overseas. From April to November 1945 he served in Manila with the General Engineer District in the Bridge Reconstruction section, first as Assistant and then as "Officer in Charge".

Van Atta has these society memberships: Tau Beta Pi, Phi Kappa Phi, Associate Member American Society of Civil Engineers

Lone Star Changes

Lone Star Cement Corp., announces the appoint of Purd B. Wright, Jr. as Manager "Incor" and Technical Service; Ernest Gruenwald as Chief Engineer, Technical Service (both are ACI members) and James F. Callery, as Manager, Publicity Department effective January 1, this year.

Awards to JPP Contributors

Awards of $50.00, $25.00 and three of $10.00 were set up by the Institute for the five contributions to the Job Problems and Practice pages of the ACI Journal for the volume year Sept. 1944 to June 1945 judged best for the period. (They are continued in the current volume year to and including June 1946). First year awards are as follows:

The top award of $50.00 to Ralph Lewis, Power Engineer in charge of Broken Hill Associated Smelters, Fort Pirie, South Australia, his contribution being one of those on the subject of "Setting Heavy Machinery on Concrete Bases" p. 362, February 1945.

The second, a $25.00-award goes to Paul A. Jones, Construction Engineer, Bureau of Reclamation, for his contribution "Precast Concrete Irrigation Pipe Jacked under Railroad Tracks" p. 705, June, 1945.

The third, fourth and fifth awards—$10.00 each—go to Walter H. Wheeler for his contribution to the Sept. 1944 JPP section; Charles Macklin for his contribution to the June 1945 Journal, and Capt. K. K. Hansen for his contribution to the JPP section in April 1945.

Definitions Pertinent to Concrete

At a meeting of Sub-Committee V, Committee C-9, American Society for Testing Materials, held in Washington January 15, 'twas voted that three revised definitions (admixture, gravel, sand) be referred to Committee C-9. The three definitions are as follows:

Admixture—A material other than portland cement, aggregate or water that is used as an ingredient of concrete, including materials added to the batch before or during mixing and materials interground with the clinker for the purpose of affecting the properties of concrete, excepting gyp-

sum used in the normal manufacture of cement.*

Gravel—The coarse granular material, larger than sand, resulting from the erosion of rock by natural agencies.

Sand—The fine granular material, usually smaller than about ¼ inch, resulting from the erosion of rock by natural agencies or from the mechanical reduction of weakly cemented sandstone. The term is often used with a qualifying adjective to denote the product of mechanical crushing, as for example, "stone sand" or "slag sand."

*See paragraph 1 "Admixtures for Concrete" reported by ACI Committee 212, ACI JOURNAL, Nov. 1944; *Proc.* V. 41, p. 73.

The program of the sub-committee includes the development of definitions for the following—curing, laitance, water segregation, bleeding or water gain, workability.

The committee is also keeping informed of the status in ASTM committee E-8 of definitions for—concrete, consistency, and plasticity.

This work of subcommittee V is reported to the Institute by the ACI representative on the committee, Dr. J. C. Witt.

THE AMERICAN CONCRETE INSTITUTE

is a non-profit, non-partisan organization of engineers, scientists, builders, manufacturers and representatives of industries associated in their technical interest with the field of concrete. The Institute is dedicated to the public service. Its primary objective is to assist its members and the engineering profession generally, by gathering and disseminating information about the properties and applications of concrete and reinforced concrete and their constituent materials.

For nearly four decades that primary objective has been achieved by the combined membership effort. Individually and through committees, and with the cooperation of many public and private agencies, members have correlated the results of research, from both field and laboratory, and of practices in design, construction and manufacture.

The work of the Institute has become available to the engineering profession in annual volumes of ACI Proceedings since 1905. Beginning 1929 the Proceedings have first appeared periodically in the Journal of the American Concrete Institute and in many separate publications. (Pamphlets presenting brief synopses of Journal papers and reports of recent years, most of them available at nominal prices in separate prints, are available for the asking.)

ACI publications in large current demand

ACI Standards—1945

148 pages, 6x9 reprinting ACI current standards: Building Regulations for Reinforced Concrete (ACI 318-41); three recommended practices: Use of Metal Supports for Reinforcement (ACI-319-42); Measuring, Mixing and Placing Concrete (ACI 614-42); Design of Concrete Mixes (ACI 613-44); and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—all between two covers, $1.50 per copy—to ACI Members, $1.00.

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer," and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944. $1.25 per copy; 75 cents to Members.

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940; 140 pages. Price $1.50—to ACI members $1.00

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages. price $2.00—$1.00 to ACI members.

Concrete Primer (Feb. 1928)

Prepared for ACI by F. R. McMillan, it had five separate printings by the Institute alone (totalling nearly 70,000 copies). By special arrangement it has been translated and published abroad in many different languages. It is still going strong. In the foreword the author said "This primer is an attempt to develop in simple terms the principles governing concrete mixtures and to show how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete." 46 pages, 25 cents (cheaper in quantity).

For further information about ACI Membership and Publications (including pamphlets presenting Synopsis of recent ACI papers and reports) address:

AMERICAN CONCRETE INSTITUTE 742 New Center Building Detroit 2, Michigan

[To facilitate selective distribution, **separate prints** of this title (42-16) are currently available from ACI without charge—quantity limited. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Aug. 15, 1946]

Title 42-16 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
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Vol. 17 No. 5 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **April 1946**

Announcement of

Proposed Manual of Standard Practice for Detailing Reinforced Concrete Structures*

REPORTED BY ACI COMMITTEE 315

ARTHUR J. BOASE
Chairman

EARL P. ALLABACH
FRANK H. BEINHAUER
RENE L. BERTIN
ROYALL D. BRADBURY
E. E. EBLING
O. W. IRWIN
FRANK KEREKES
T. D. MYLREA
FRED L. PLUMMER

also with the cooperation of the
CONCRETE REINFORCING STEEL INSTITUTE

FOREWORD

What follows is not the report of Committee 315. It is an announcement about the report. The Committee's report exceeded the possibilities of the format of ACI Journal pages. It will be issued as an "ACI special publication" with a much larger page, including still larger folded inserts for adequate reproduction of numerous drawings—all in stiff covers with spiral binding—to lie flat when open at any page. It has been predicted that it will be needed not only by the designing engineer but by every draftsman on reinforced concrete. * * * * Page proof corrections are in the printer's hands as this Journal goes to press. Anticipating that it will be mailed (to all ACI members in good standing and to all ACI membership applicants as of the month of mailing†) before the issuance of the June Journal, this announcement is made to invite discussion of the report—discussion which may influence Committee 315 action before the ACI Standards Committee is asked to release the report in final form for presentation to the Institute on a motion for adoption as an ACI Standard. Such discussion (copies in triplicate) should reach the office of the ACI Secretary by August 15, 1946.

ANNOUNCEMENT

A "Proposed Manual of Standard Practice for Detailing Reinforced Concrete Structures" has been completed by Committee 315 after several

*As released by the ACI Standards Committee for publication and discussion.

†The proposed ACI manual will be available to non-ACI members at $2.50 per copy; somewhat cheaper in quantity. The Institute's initial distribution will be in consideration of ACI membership; additional copies to ACI members at appropriate Member and quantity discounts.

years of intensive work. This Manual will be of assistance to the designer, detailer and student.

It has been the purpose of Committee 315 to present improved methods and standards for preparing drawings for the fabrication and placing of reinforcing steel. The use of these improved methods and standards will result in better reinforced concrete construction and also simplify and reduce the amount of work ordinarily required to prepare both engineering drawings and placing drawings.

The fundamental purpose of "engineering drawings" is to show the designer's ideas so that the structure can be built in accordance with his calculations and ideas. These drawings must give the information necessary for building forms and placing concrete and for preparing the "placing drawings" (sometimes called "detail drawings"). The "placing drawings" must contain the information necessary for fabricating the reinforcement and placing it in the forms.

There has been little uniformity in the preparation of these drawings with a consequent great waste of effort in interpreting them and frequent misunderstandings. Ambiguities and omissions are much more frequent in the "engineering drawings" than in the "placing drawings." The latter are usually made by the fabricators who have fairly uniform practice within their organization although there is considerable variation among different fabricators. Uniformity between these organizations will aid the designer in checking the "placing drawings" and the steel setter in placing the steel.

Important parts of the Manual include:

Drawing standards

Standards and comments on size of sheet, layout, scales and symbols.

Marks

Discussion of various sytems of marking members and reinforcement.

Schedules

Discussion of various types of schedules and recommendations for their use.

Engineering drawings

Information required on drawings and best methods of showing it.

Placing drawings

Best methods of showing necessary information.

Fabricating detail standards

Standards for bending, hooks, stirrups, typical bar bends, slant lengths, beam widths, spirals, ties, column verticals, splices and dowels.

Notes to designers and detailers

Attention is called to points which frequently are overlooked or which cause trouble. Conformance with the suggestions given eliminate most difficulties of interpretation, fabrication and construction.

Shop procedure

Brief description of steps in the shop.

Warehouse stock

Notes material usually carried in stock.

Tolerances

Standard tolerances in dimensions of fabricated reinforcement.

Extras

Basis for extra charges for material and fabrication.

Welded wire fabric

Data on fabrication, size limitations, detailing and common styles.

Accessories

Standard accessories and their use.

Typical drawings

Twenty-one typical engineering and placing drawings are shown for various structures to illustrate the use of the standards and methods advocated in the Manual. A short discussion of the important points accompanies each drawing. The drawings show: concrete joist floor, flat slab floor, beam and girder floor, two-way slab floor, foundations and columns, architectural concrete building, walls, stairs, circular tank and rigid frame bridge.

The complete "Table of Contents," "List of Drawings" and "List of Tables" follow:

TABLE OF CONTENTS

INTRODUCTION

I. ENGINEERING AND PLACING DRAWINGS — Page

1. General 6
2. Drawing standards 6
3. Marks 7
4. Schedules 8
5. Engineering drawings 8
6. Placing drawings 8
7. Fabricating detail standards 9
8. Bar lists 12
9. Notes to designers and detailers 16

II. FABRICATING SHOP PRACTICE

1. Shop procedure 17
2. Warehouse stock 18
3. Tolerance 18
4. Extras 18

III. WELDED WIRE FABRIC

1. General 19
2. Manufacturing limitations . . 19
3. Commercial practice 19
4. Rolls and sheets 19
5. Laps 19
6. Detailing 19
7. Styles of fabric 20

IV. ACCESSORIES 22

V. TYPICAL DRAWINGS 32

LIST OF DRAWINGS

Description	Page
Recommended layout for drawings	6
Splice details	13
Composite construction details	14
Typical bar list	15
Details of bar tags	17
Tolerances	18
Detailing laps of wire fabric	20
Detailing flat sheets of wire fabric	21
Standard Accessories	23
Standard number and location of accessories	24
Standard column ties	26
Typical bar bends	27
Standard hook details	28
Slant diagram	31
Concrete joist floor—	
—engineering drawing	33
—placing drawing	34
Flat slab floor—	
—engineering drawing	37
—placing drawing	38
Beam and girder floor—	
—engineering drawing	39
—placing drawing	40
Two-way slab and beam floor—	
—engineering drawing	41
—placing drawing	42
Foundations and columns—	
—engineering drawing	43
Foundations—	
—placing drawing	44
Columns—	
—placing drawing	45
Architectural concrete building—	
—engineering drawing	46
—placing drawing	47
Wall details—	
—engineering drawing	48
—placing drawing	49
Stair detail—	
—engineering drawing	50
—placing drawing	51
Circular tank—	
—engineering drawing	52
—placing drawing	53
Rigid frame bridge—	
—engineering drawing	54
—placing drawing	55

LIST OF TABLES

Manufacturing limitations for welded wire fabric	20
Common styles of welded wire fabric—	
—one way type	21
—two way type	21
Accessory specifications and standard nomenclature	22
Maximum spacing of column ties	25
Maximum number of column bars	25
Lap splices	25
Minimum beam widths	29
Slants and increments for 45 degree bar bends	30

To facilitate selective distribution, **separate prints** of this title (42-17) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Aug. 15, 1946.

Title 42-17 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

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Vol. 17 No. 5 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **April 1946**

Maintenance and Repair of Portland Cement Concrete Pavement*

By A. A. ANDERSON†

SYNOPSIS

Highway maintenance consists of routine maintenance which is carried on daily and special maintenance conducted at appropriate intervals with specially trained crews—the better the routine maintenance, the less generally, the special maintenance.

Routine maintenance operations consist largely of sealing cracks and joints against infiltration of dirt and water and maintaining that seal. Operations vary with types of joints and climatic and subgrade conditions. Maintenance procedure for expansion joints filled with non-extruding and extruding material, contraction joints, construction joints and cracks, is described.

Items of special maintenance are covered in considerable detail as they generally require more engineering supervision.

Patching concrete pavements with concrete is best because, when properly done, patches are integral with the pavement and not inferior to the original slab. Procedure and methods of construction, based on extensive experience records, are discussed under the headings: slab thickness, removal of old slab, preparation of subgrade, materials and proportions of concrete, finishing and curing.

"Mudjacking" and materials and mixtures for the slurries are outlined as a means of both raising settled slabs and minimizing and preventing damage from pumping slab ends.

Methods and means of protecting existing concrete pavements against surface scale where air-entraining portland cement was not used during construction are also outlined.

INTRODUCTION

The maintenance and repair of concrete pavements may be divided into two general classifications: routine and special. Routine maintenance is handled by a fixed patrol crew who perform the day to day

*Received by the Institute Jan. 21, 1946.
†Manager, Highways and Municipal Bureau, Portland Cement Association, Chicago, Ill.

operations to keep the road in good condition. Special maintenance requires special crews, or, at least, special equipment for major repair jobs made necessary by subgrade failure, errors in the original construction or overloading of the slab with illegal truck and axle loads.

ROUTINE MAINTENANCE

The routine maintenance of concrete pavement is not difficult, nor does it require special qualifications or high skill. Ordinary workmen, properly supervised, can perform all the operations required. Neglect of routine maintenance, however, may cause expensive special maintenance operations. Efficient and timely maintenance is not only good economy but it keeps the pavement sightly and attractive.

Joints and cracks

An important item of routine maintenance is the sealing of joints and cracks against infiltration of dirt and water. The type of filler and sealing material used in construction of expansion joints has a direct influence on methods and materials used in their maintenance. Joints in earlier pavement may have material of the extruding type while those in pavements built later are likely to have non-extruding material. The most commonly used types of non-extruding fillers are treated fiber and boards. Other non-extruding fillers are cork, sponge rubber, air cushion and mixtures of sawdust or cottonseed hulls with asphalt.

Non-extruding expansion joints. When non-extruding fillers are found, maintenance consists usually of repairing or replacing the seal, which occupies the top ¾ to 1½ inches of the joint. The maintenance of the seal should not be delayed until it has disappeared. Joints should be inspected occasionally and when it is found the sealing material has loosened or become hard and lifeless it should be replenished or replaced.

When the joints require only replenishing of the sealing material, it is good practice to remove all loose and foreign material from the joint before resealing. When the sealing material needs replacement, all of it should be removed to give good bond for the new material.

The removal of the old seal frequently can be done with stiff fiber or steel brooms and the use of hand-operated bars shaped to fit the joint space. Special type plows are often used for removing old joint sealing material. (Fig. 1.)

The concrete must be thoroughly dry and the joints completely cleaned, before applying new material. Compressed air can be used to good advantage as a final cleaning and drying operation.

Bitumen used for sealing should be poured with a pot which has a relatively small spout (Fig. 2a and 2b) so small quantities can be placed as required. The material used to fill deep cracks or joints will shrink

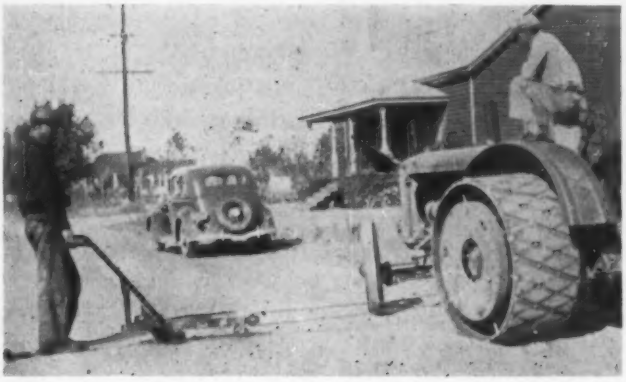

Fig. 1—Winch on tractor pulls a special plow for removing old sealing material.

perceptibly while cooling. In this case the crack or joint should be refilled.

Shortly after pouring, the sealing material is dusted with fine sand or rock dust to prevent it from being picked up by traffic. This also helps to eliminate unsightly marks on the pavement.

Extruding type expansion joints. Where expansion joints have been filled with an extruding type filler, the material forced out on the slab causes bumps which are most objectionable to traffic. The extruded material can be trimmed or cut away with a shovel or a plow can be used to trim the material away slightly below the level of the pavement surface. If required to reduce infiltration of water and inert material, the joint may be resealed with a suitable sealing material.

Contraction joints. These are usually of the dummy groove type, the groove being formed while the concrete was plastic and later filled with

Fig. 2a (left)—Sealing Cracks.
Fig. 2b (right)—Sealing longitudinal joints.

a poured sealing material. They are also constructed by inserting a premolded bituminous strip in the freshly placed concrete.

The groove type may be maintained in the same manner as described for an expansion joint with a seal over a non-extruding material.

The premolded strip should have been placed at or near the surface of the pavement and at right angles to the surface. When so installed it can be cleaned and sealed if necessary. If the strip was set some distance below the surface and a clear space was not edged above the material during construction, some spalling may have developed. In these cases, it is desirable to remove any loose material and the concrete fins above the strip and then seal the joint.

Construction joints. These may be either of the butt or tongue-and-groove type. From a maintenance standpoint they do not differ from contraction joints and the procedure of maintaining them is the same.

Hinged joints. These are usually longitudinal joints and the slabs are tied together with steel reinforcement or tie bars. They may be formed with the use of a deformed metal plate staked to the subgrade or with a surface groove as are contraction joints. They are also maintained with the same methods as described for contraction joints.

Cracks. Narrow cracks are maintained similarly to contraction joints of the strip type. Wide cracks should be well cleaned as are joints before sealing.

The difficulty of maintaining seals at cracks is a major reason for favoring the use of contraction joints at intervals which will control cracking. Most of the material used to seal a crack is on top of the pavement. In cold weather, when the seal is most needed, bituminous materials become hard and brittle and their tendency to adhere to the concrete is reduced. In this condition they are likely to be removed by traffic. In contrast, the seal in a contraction joint, especially of the preferred dummy groove type, is below the surface and is much less likely to be displaced by passing wheels.

Sealing materials

Formerly about the only materials used for sealing joints and cracks were asphalts and tars. These bituminous materials were far from satisfactory. Those which were not so soft that they become sticky or flowed from the joint in warm months would be too hard and brittle and lose their adhesive properties in cold weather. In an effort to get more uniform properties throughout the year, fine material, such as diatomaceous earth, has been mixed with asphalt. This has allowed the use of asphalt so soft that without the addition it would flow from the joint in the summer but the softer material retains some plasticity in the winter.

Recently chemicals having special properties have been added to asphalts to produce fillers which, it is claimed, will have better adhesiveness and will remain plastic and non-fluid over a wider range of temperatures. It remains to be demonstrated how well these new fillers will perform under service conditions.

An increasing number of non-bituminous joint sealing materials are appearing on the market. Using natural or synthetic rubber, their development has been retarded by the war-time shortages, but they are expected to become active in post-war years. More costly than bitumens, they must give longer service to be economical but there is evidence that if properly placed they will perform well. Because the range of temperatures at which they can be placed without damage is fairly narrow, special equipment for heating and applying them seems desirable. Similar equipment would be desirable also with bituminous sealing materials.

There is plenty of room for improvement over the bituminous joint sealing material as used in the past. Both bituminous and non-bituminous materials give promise of much better performance than anything we have known heretofore.

Repair of spalls

Spalling at joints is caused usually by defective construction practices; such as finishing concrete over the expansion joint space, not finishing groove directly above expansion material, lack of continuity of expansion joint filler so that a portion of the expansion space becomes filled with concrete, placing premolded dummy groove strip too far below the surface or not at right angles to the surface and using mortar of low quality to finish joints or edges. The condition which causes the spalls must first be corrected by the maintenance crew.

If the area to be restored is not over 4 to 5 inches from the joint and quite small it may be filled with the sealing material for the joint. Larger areas should be repaired with concrete.

The Department of Highways of the District of Columbia has obtained excellent results in repairing spalled areas with portland cement mortar. The method was originally reported in *Roads and Streets*, March, 1941.

The area is cleaned thoroughly and a 25 per cent solution of muriatic acid applied. The acid is then washed out completely and a 1:2 mortar placed in the hole. This mortar has a low water-cement ratio and calcium chloride is used as an accelerator.

The patch is struck-off, edged and finished to match the surface texture of the adjacent slab. It is then cured 4 to 8 hours, as all patches are placed in the morning and opened to traffic at the close of the work-

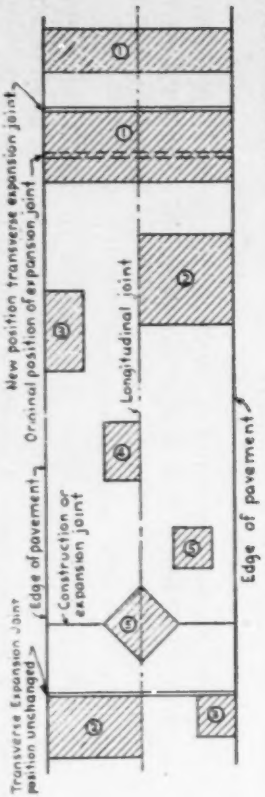

Fig. 3 (left)—Types of patches classified according to their position in the pavement.

1 Full width patch; see Fig. 4
2 Half width, or single lane patch
3 Exterior edge patch
4 Interior edge patch
5 Interior or plug patch

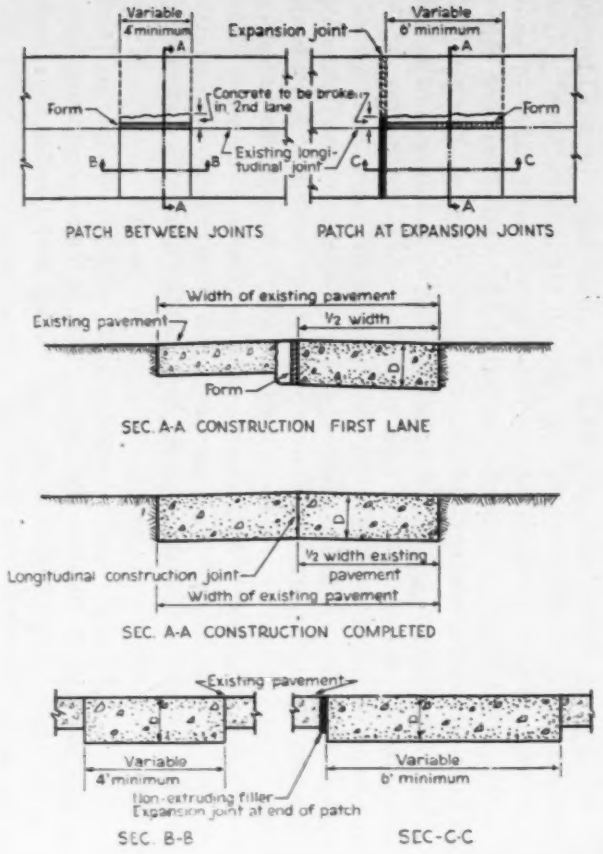

Fig. 4 (right)—Patching portland cement concrete pavement—Type 1.

ing day. It is felt that the loss of an occasional patch is preferred to the use of barricades or other obstructions to a clear roadway during the night.

Many of the patches placed in the district are less than 2 inches thick. They presented a good appearance 3 or 4 years after placement, with no sign of failure.

SPECIAL MAINTENANCE

Patching

Patching of concrete pavement may be routine or special maintenance depending on the amount to be done. In either case the general procedure is the same.

Concrete patches are easily placed and, if properly done, become an integral part of the pavement, in no way inferior to the original portion of the slab. Patches are generally classified into five types according to their position in the pavement. Fig. 3 shows the location of each type of patch, while Fig. 4 gives the detail concerning one type. Each type has been discussed in detail in *Wartime Road Problems Bulletin No. 6** of the Highway Research Board and will not be considered further in this paper. The procedure in placing patches is practically the same regardless of the type into which it falls.

Slab thickness. The original pavement thickness must be adequate for the loads it is carrying; otherwise, the advisability of a patching program may be open to question and reconstruction or resurfacing should be considered.

Local unsatisfactory subgrade conditions should be corrected before placement of patches. When this is done, the thickness of patches should be:

Original Pavement	Depth of Patch
Thickened Edge Slab	D = 1.3 times center thickness original slab for unprotected corners D = 1.2 times center thickness original slab for patches not involving unprotected corners
Uniform-Thickness slab with dowels or other load transfer devices at expansion joints	D = 1.1 times original slab thickness for all patches involving unprotected corners D = same depth as existing slab for all other patches

The depths from the above table are for relatively small areas. If pavements are to be replaced for the full slab widths for lengths of 50 feet or more the original design of the cross section may be followed.

Removal of old slab. Where small areas are to be patched, hand methods for removing the old pavement may be used. Where patches

*Patching Concrete Pavements with Concrete.

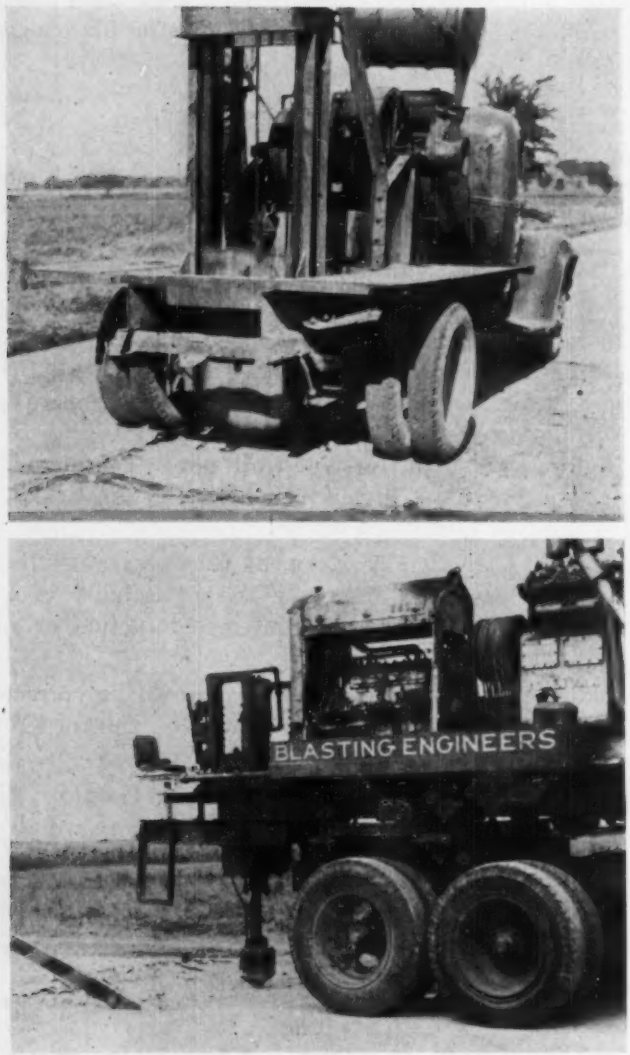

Fig. 5—Paving breakers: (top)—drop type; (bottom)—hammer type.

have fairly large areas and are close together, mechanical means should be used to break the pavement. There are several types of pavement breakers on the market (Fig. 5) some of which drop a weight on the pavement while others use compressed air to drive jack hammers to break the slab.

The edge of the patch should be as vertical and straight as possible. Acute angles should be avoided at all corners. Except for the diamond-shaped patch, all edges are parallel to or at right angles to the centerline and edges of the original slab.

Preparation of subgrade. Local conditions of the subgrade, such as frost boils or frost heave, seepage from a water-bearing strata of soil, or a number of other conditions may have caused the breakage. The conditions should be corrected before the concrete for the patch is placed.

All loose concrete should be cleaned from the old pavement and the patch area. The edge of the original slab and the subgrade should be damp but not wet at the time of placement of concrete.

Materials and proportion of concrete. The materials for the concrete and the proportions of the mix should comply with the specifications of the city or state highway department placing the patch. It is recommended that patches be kept closed to traffic until the concrete has attained a modulus of rupture of 500 psi except in cases of emergency when they may be opened earlier if possible damage to an occasional patch may be justified by demands of traffic. By use of high early strength concrete it usually will be possible to open a patch in 24 to 72 hours depending on temperature, concrete mix used and other conditions.

High early strength concrete. There are three methods available for producing high early strength concrete:

1. *Use of low water-cement ratios with normal portland cement* which, practically, means richer mixes to obtain the required workability. Water-cement ratios should not be less than 4 gallons per sack of cement to avoid difficulties of placement, curing and excessive volume changes. The design of mix is best based on tests of materials to be used but in the absence of such tests the curves on the left side of Fig. 6 may be used as a guide to design the mixture.

2. *Use of high early strength portland cement.* As in 1, the water-cement ratio should not be less than 4 gallons per sack of cement. Until data on local materials can be secured, the design of the mixture may be guided by the curves on the right hand side of Fig. 6.

3. *Use of calcium chloride as an accelerator.* Not more than 2 per cent of calcium chloride by weight of the cement may be added to concrete made of either normal or high early strength portland cements to increase the early strength. Data on the effect of calcium chloride on flexural strength are quite limited, but such as are available indicate that an addition of 2 per cent may increase the 1-day strength of concrete made with normal portland cement about 160 psi and that of concrete made with high early strength portland cement about 110 psi when the water-cement ratio in either

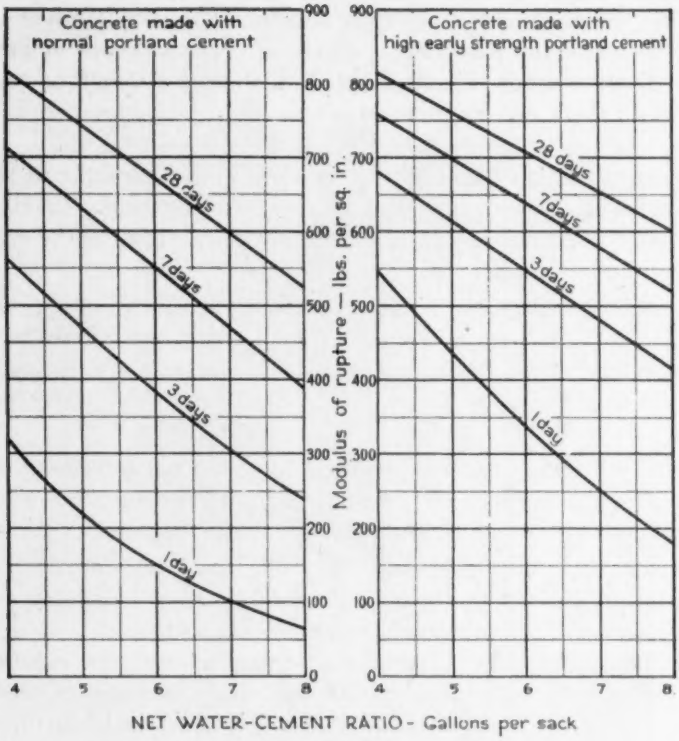
Concrete made with normal portland cement
Concrete made with high early strength portland cement
28 days
7 days
3 days
1 day
Modulus of rupture — lbs. per sq. in.
NET WATER-CEMENT RATIO - Gallons per sack
Flexural strength of 6"x6"x30" beams — ⅓ point loading
All specimens cured saturated atmosphere at 70°F. - tested damp.

Fig. 6—Flexural strength—water-cement ratio relation for concrete made with normal and high early strength portland cement.

concrete is 4 to 6 gallons per sack and the concrete is cured at normal summer temperatures. The 3-day flexural strengths of either concrete will be increased about 50 psi while there will be little or no increase at 7 and 28 days.

It is suggested that the above data be used as a guide only until tests can be made on the materials and mixtures in use.

Mixing and placing concrete. It is preferable that the aggregates and cement be proportioned by weight. This is common practice now and portable scales are available for use where a central proportioning plant is not used. The concrete should be machine mixed for at least one minute if transit or central mixing is not used. Transit and central mixing have been found very useful where the size of the job warrants their use (Fig. 7). The concrete must be placed and finished without the addition of more water.

The concrete as first placed is struck-off and tamped at an elevation slightly higher than the finished surface as concrete shrinks slightly

as it hardens. The effect of shrinkage can be minimized by a second tamping a short time after placement. Mechanical vibrating equipment may be used advantageously to consolidate the concrete, especially at the edges of the patch.

Finishing. Patches should be finished to have a surface texture similar to that of the adjacent old concrete; therefore, the method of finishing should be the same as that used on the original slab. All joints and outside edges of the patch should be edged with a ¼ inch edger.

Curing. Patches may be cured most advantageously with curing membrane (Fig. 8) or impervious paper. Usually it will be inconvenient and uneconomical to use any method involving frequent or continued

Fig. 7—Placing concrete in patch from transit mixer.

Fig. 8—Applying curing membrane to patch.

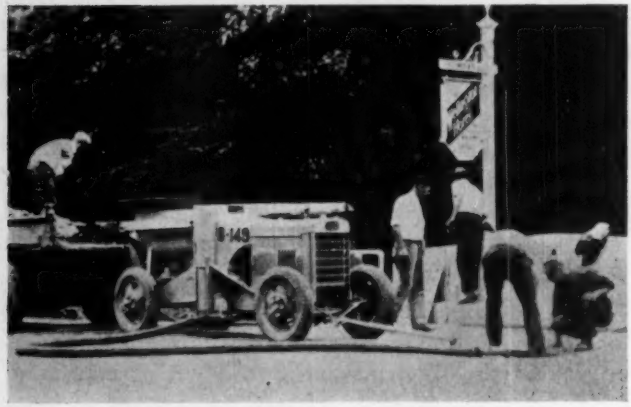

Fig. 9—A large mudjack machine such as would be used on rather extensive projects. The truck at the left has a load of materials for the "mud." Note that the pavement has not been closed to traffic during the operations.

use of water as the patches are too scattered. Except in emergency, curing should be continued until the patch is opened to traffic.

Mudjacking

Concrete pavement slabs can be raised satisfactorily and economically by mudjacking. This method may be used to correct surface irregularities, improve riding qualities, reduce impact from fast moving traffic or to correct faulty drainage in paved areas where water does not flow properly to drainage inlets.

Mudjacking is used also for the correcting of subgrade conditions which result in pumping at transverse joints—discussed later in this paper. The Highway Research Board bulletin, *Maintenance Methods for Preventing and Correcting the Pumping Action of Concrete Pavement Slabs*, outlines the procedure used in detail.

The method consists of mixing earth, water, portland cement and bitumen in such proportions as to form a mud or slurry which is then pumped through holes drilled in the pavement. The mud flows under the slab and exerts sufficient pressure to raise it. The actual pressure required is small as an 8 inch concrete pavement weighs only about 0.65 psi.

Composition of the slurry varies according to the practices found best in each state and whether it is to be used for correcting irregularities or to prevent pumping. When it is used to adjust the elevation of the pavement, the slurry usually contains no bitumen. In this case about 1 sack of cement is used per cubic yard of loam with the water varied with the moisture content of the loam but just enough to let the mixture flow.

The field procedure in mudjacking is substantially the same in raising slabs as for correcting pumping action, therefore it will be discussed under that topic.

When properly used the mudjack (Fig. 9) provides a convenient, efficient and economical means for raising concrete pavements. Costs depend on numerous variables such as distance slab is to be lifted, area of pavement involved and other local conditions. However, costs should always be less than levelling with bituminous mixes and results should be better as well as avoiding disfigurement of the pavement by blotches of material of a contrasting color.

One highway district in Texas has experimented with hot asphalt as a mudjack material using the asphalt distributor as the machine. The distributor will supply a pressure of 15 to 30 psi which is ample for the method.

A double line of hose is used from the distributor to the nozzle, one carrying the asphalt under pressure and the other, a returning hose which is used to keep the asphalt circulating so that it will not "freeze" in the hose. To protect workmen in case of breaking hose, the pressure hose is incased in a fire hose.

As soon as any cavities under the pavement are filled and the slab brought to grade, a tapered wooden peg is driven into the mudjacking hole to keep the asphalt from welling out.

Detailed costs are not available but it is believed to cost less than the usual mudjacking process as very little equipment and few workmen are required. This method has not been in use long enough to give definite results but it appears to have merit.

Pumping of slabs. Pumping of concrete pavements may be defined as the ejection of a soupy mixture of subgrade soil and water from the joints and cracks in the pavement as the slab ends are depressed by heavy wheel loads. This condition became increasingly serious during the war due to the increase of heavy truck traffic, much of which exceeded legal load limits.

Pumping is caused by an unfavorable combination of subgrade soil type, free water between the pavement and subgrade, and the number and weights of loads traveling the pavement. When any one of these items is absent, pumping does not develop.

Subgrades conducive to pumping usually consist of soils containing a predominance of silt and clay. Non-plastic granular soils such as sands and gravels are not susceptible to pumping.

Free water is largely surface water that has infiltrated through cracks, joints and the edges of the slab. As a result pumping generally takes place during periods of prolonged wet weather such as may be expected in the spring and fall seasons.

Good maintenance will reduce pumping considerably. Where joints and cracks have been well cleaned and sealed and ruts kept out of

shoulders at slab edges, pumping has been reduced materially or eliminated.

Where pumping has developed steps should be taken to stop the action and restore the pavement to the original grade line. Early attention is most effective and may preclude subsequent structural damage. Mudjacking has been successfully used in stopping pumping and in eliminating rough riding joints and subsequent breakage.

Mixtures for mudjacking. The slurry used varies with the location in which the work is done and according to the soil available. A suitable soil must slake readily in water to form a mixture of uniform consistency and be reasonably free of organic material and gluelike colloids.

Illinois uses a slurry composed of 1 part by volume of portland cement, 1 part limestone dust, 1.75 parts slow curing liquid asphalt (SC-2 grade) and 5.5 parts soil, with about 3.25 parts of water. Kansas uses a mixture of 8 parts by volume of portland cement, 5 to 8 parts of MC-2 or MC-3 cutback asphalt and 84 to 87 parts of soil with just enough water to produce a thick mixture.

In Indiana the two mixtures showing best results were: 1) 16 per cent portland cement, 7 per cent bitumen (RC-3) and 77 per cent soil and 2) 17.5 per cent portland cement, 3.5 per cent bitumen (tar) and 79 per cent soil.

The most recent performance survey of an experimental section showed that mudjacking had been successful in reducing the expected number of cracks.

Ohio found an oil asphalt filler having a penetration of 30 to 45 at 77 F satisfactory for filling the voids under pumping concrete slabs. The material was generally forced under the pavements by means of the hand spray equipment of a standard bituminous distributor through holes drilled in the pavement with a standard jackhammer and drill.

Missouri reports to the Highway Research Board that it has been successful generally in stopping pumping with a slurry composed of 4 sacks of cement per cubic yard of top soil and 50 to 55 per cent water.

Field procedure. A hole is drilled for the nozzle of the mudjack near the junction of the transverse joint or crack and the center joint. The location should be about 10 inches from the transverse joint or crack and the longitudinal joint and on the far side of the joint in the direction of traffic.

When the pumping action has caused settlement of the slab, the holes should be drilled 30 inches each way from the intersection of the cracks or joint with the outside edge of the slab.

Filling the voids and raising the slab require the same initial procedure. All the water trapped under the slab is forced out with com-

pressed air. To allow the escape of water, the joint should be vented at the outside edge of the slab. It may be necessary also to remove some of the joint material to allow all water to get out.

Next the slurry is pumped through the hole until it appears in the crack or joint and at the pavement edge. Shots of air may be alternated with shots of slurry to force the mix the maximum possible distance.

When the slab is to be raised, the vent at the edge of the pavement is closed and mudjacking continued until the slab is lifted to the desired elevation. Otherwise, pumping is stopped when the mix appears at the edge.

The holes are finally filled with clay to seal the mix under the slab. Later, they should be filled with a bituminous mixture.

Mudjacking to be successful requires careful engineering supervision and skilled operators. The best method is to train a crew from headquarters for the work and to send it wherever mudjacking is necessary. Inexperienced workers may not be able to stop pumping at slab ends and quite possibly will cause severe damage to the pavement.

Surface Scale Due to Salt or Other Action

Surface scaling of concrete pavements may be caused by poor materials, imperfect workmanship in finishing or by the continued application of salts to remove ice. The latter is especially likely to cause scaling during the first two or three years after construction. Air-entraining cement used in pavement construction is highly effective in preventing this type of surface scale.

Protective treatment. Surface scale by salt action in pavement not constructed with air-entraining cement may be minimized or prevented by protective treatment. The treatments recommended are two applications of boiled linseed oil thinned with kerosene, naptha, mineral spirits or turpentine.

The oil mixtures are applied very thinly and in order to facilitate application on a large scale the thinned boiled linseed oil is sometimes emulsified and mixed with sufficient water to increase its volume so that it may be applied with a standard bituminous distributor. The first application is made at the rate of about 50 sq. yd. per gallon of oil and thinner and the second at the rate of 65 to 70 sq. yd. per gallon of oil and thinner. Enough water is added so that there is about 10 gallons of emulsion for one gallon of oil and thinner. The pavement should be clean and dry when the material is applied with the air temperature not less than 50 F and higher temperatures preferred. The best time to apply the mixture is before the project is open to traffic and the pavement is warm and dry.

The protective treatment will not stop surface scaling once it has secured an appreciable start but it is effective if applied in time.

Thin concrete patches. New Jersey has a record of successful placement of thin concrete patches for the last 18 years. The State Highway Department removes all unsound concrete to a uniform depth of not less than 1 inch. The area is then cleaned with compressed air, dampened, dusted carefully with neat cement and concrete placed. A mix of 1:1:2 is used with largest sized aggregate not over ¾ inch. The fresh concrete is thoroughly tamped and screeded slightly high. After one to two hours the concrete is retamped and the patch finished to grade.

Shotcreting of scaled areas has been tried. In Chicago, Illinois, it was applied to the scaled areas, finished and immediately opened to traffic without any curing period at all. About a month after placement some 25 per cent of the patching material had been loosened by traffic. It is believed that this method might have merit if the patches were protected and cured thoroughly and good bond secured between the patch and the old slab.

Scaled areas may also be patched with bituminous mixtures but this results in unsightly pavements due to the blotched appearance of dark material on a light colored pavement. If the scaled areas are sufficiently numerous to warrant resurfacing the entire pavement, thin bituminous coverings of ¾ inch or less have been found successful by many state highway departments.

CONCLUSION

The major item of good maintenance is good operating procedure under competent supervision. Concrete pavement can be kept in good condition for traffic at very low cost if it is not neglected and required maintenance performed properly and at the right time.

To facilitate selective distribution, **separate prints** of this title (42-18) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Aug. 15, 1946.

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Curing Concrete with Sealing Compounds*

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Members American Concrete Institute

SYNOPSIS

Following a general discussion of the principles involved in the curing of concrete, methods are presented for the effective use of sealing compounds for this purpose. Included are data for evaluating concrete curing by membrane treatment, in terms of equivalent moist curing. The paper discusses preferred methods of curing by use of sealing compounds, and outlines a specification and acceptance test for their purchase.

FACTORS IN PROPER CURING

The benefits obtained by properly curing concrete after it has been placed are fully appreciated by engineers. However, for the purpose of introducing the subject of curing with sealing compounds, a restatement of some of the general principles of curing is appropriate.

Fundamentally, curing is the maintenance of moisture within, or supply of moisture to, immature concrete so that hydration of the cement will be as complete as it is practicable to obtain. Although concrete of the usual plastic consistency contains more water than is required for complete hydration, it appears that not all of it can be readily utilized by the cement. If any of the mixing water is lost by evaporation while the concrete is in its infancy, such loss detracts from the water immediately available to the cement, with the result that the orderly development of its hydrated structure is interrupted. Concrete which is continuously maintained in a moist condition improves in strength, hardness, impermeability and other qualities. The degree of improvement is related directly to the extent that the concrete is kept moist. (1), (2), (3). While the usual method of accomplishing this is to supply

*Presented 42nd ACI Convention, Feb. 20, 1946.
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1"Study of Methods of Curing Concrete," by H. F. Gonnerman, ACI *Proceedings* Vol. 26, 1930, p 359.
2"The Moist Curing of Concrete," by H. J. Gilkey, *Engineering News-Record* Oct. 14, 1937, p 630.
3"Concrete Curing Compounds," by H. S. Meissner and S. E. Smith, ACI *Proceedings* Vol. 34, p 549.

additional water by sprinkling, ponding, or covering with wet burlap, sand or earth, it may also be done by sealing the surfaces of the concrete to prevent evaporation and so preserve the original mixing water.

The most critical periods for concrete are the first few hours, followed by the first few days immediately after placement. During this interval the rates of reaction between cement and water are greatest and the need for curing most essential. Therefore, to achieve the greatest effectiveness, curing should begin immediately after the concrete has been placed or, in the case of finished surfaces, as soon as the finishing operations are completed. Having begun, curing should be continued without interruption for some minimum period that will insure developing the potential qualities of the concrete to the greatest practicable degree.

Concrete that is exposed to the weather, that is backfilled with moist earth or placed upon a moist subgrade will not dry as completely as structural concrete under a roof. Curing in the latter, if it has been accomplished by the application of water, ceases when this treatment stops and the concrete dries out. However, in the other cases it will be prolonged to the extent that moisture remains in the concrete or is available to it. For many years, perhaps indefinitely, and although the concrete at times may dry out, hydration will resume whenever moisture becomes available, as from rains and condensation.[4] In many cases, particularly with slabs, additional moisture is supplied from the subgrade.[5] Thus nature plays a part in providing the ultimate in curing, but it remains for us to see that concrete gets a fair start through the early, critical period, when artificial curing is needed.

In these days, when speed of construction is a paramount issue, contractors and engineers are turning more and more to the use of liquid sealing compounds as a means of curing. In many sections of the arid west, water for curing must be transported under difficulties for many miles and its scarcity has popularized the alternate method of curing with membrane compounds. As the term implies, liquid sealing compound is applied to the exposed surfaces of concrete as a liquid, and, upon evaporation of the volatile portion, leaves a coating that more or less seals the concrete surface against the evaporation of mixing water. Theoretically, this contained water is ample for complete hydration of the cement and, if retained, might be expected to accomplish satisfactory curing. Actually, thorough curing is not obtained unless additional moisture is supplied. However, it has been fairly well established that an efficient sealing compound, properly handled, does retain sufficient water under most conditions to provide curing equal to many days of continuous water cure; this is the principle upon which membrane curing

[4] "Concrete Performance in an Arid Climate," by T. E. Stanton, *Proceedings* ACI, Vol. 37, p 141.
[5] "Measurement of the Moisture Content of Concrete," by R. W. Spencer, ACI *Proc.*, Vol. 34 (1938), p 45.

is based. Membrane curing also avoids the injurious rapid surface drying of the concrete which suddenly follows the cessation of water curing. It extends the drying period and maintains a more uniform moisture content throughout the concrete than obtains when water curing is abruptly ended.

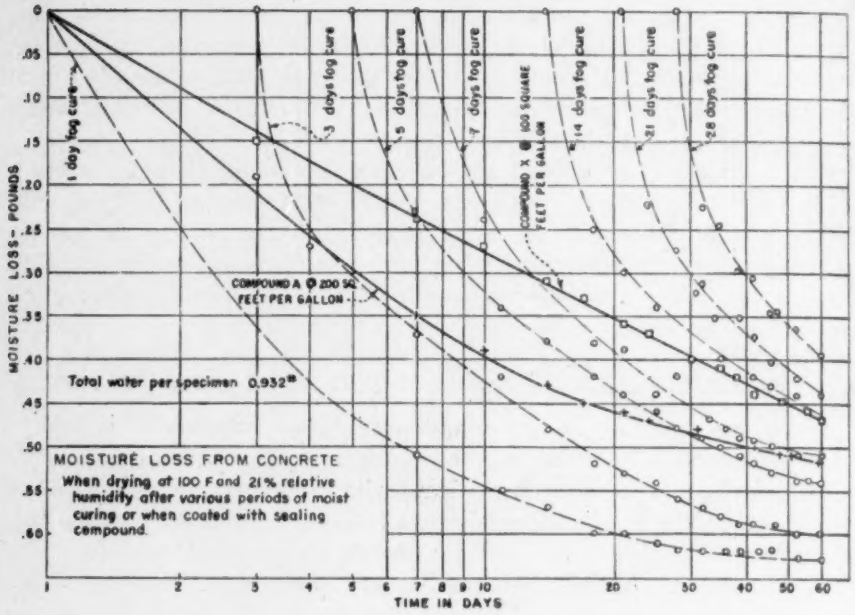
It is obvious that a coating of sealing compound should possess a high order of resistance to the passage of water vapor. It is equally apparent that the compound should be applied to the concrete before any appreciable amount of the mixing water has had an opportunity to evaporate. These are prerequisites for the effective use of liquid sealing compounds in curing concrete.

EVALUATING THE CURING AFFORDED BY SEALING COMPOUNDS

Since moist curing is the accepted standard practice, it is logical to compare the results of membrane curing with those secured under this established method. Most investigators have adopted this premise, but the criteria by which they have judged the results have been varied and devious. In this paper the authors present a method of procedure and evaluation which utilizes moist curing as the standard for comparison, with moisture retention and elastic modulus of the cured concrete as yardsticks of performance. Modulus of elasticity, determined by vibratory methods, has within late years been recognized as a simple and reliable indicator of the general quality of concrete and it has been found to be very sensitive to the amount of moist curing given to the concrete.

Concrete bars, 3 x 3 x 15 in., are cured for 3, 5, 7, 14, 21, and 28 days in a standard 70 F fog room, and at the end of these several periods are subjected to drying at 100 F and 21 per cent relative humidity. The latter conditions were selected as the simplest representation, within the laboratory, of the type of exposure which concrete endures during summer months in the arid west. Twenty-one per cent relative humidity is the humidity resulting within a tight enclosure, held at 100 F, in the presence of a saturated solution of calcium chloride with excess crystals in the solution, and likewise approximates the average atmospheric moisture conditions in arid regions of the west. As the bars dry, they are periodically weighed and vibrated to dynamically determine their elastic modulus. The results of such tests are portrayed in Fig. 1 and 2, which show moisture loss and accompanying reduction in dynamic modulus of elasticity to sixty days age.

Companion bars are coated with sealing compound at twenty-four hours age, when the specimens are removed from their molds, and are placed immediately in the 100 F, 21 per cent humidity atmosphere. Observations for moisture loss and dynamic elastic modulus are made


MOISTURE LOSS - POUNDS
TIME IN DAYS
1 day fog cure
3 days fog cure
5 days fog cure
7 days fog cure
14 days fog cure
21 days fog cure
28 days fog cure
COMPOUND X @ 100 SQUARE FEET PER GALLON
COMPOUND A @ 200 SQ. FEET PER GALLON
Total water per specimen 0.932#
MOISTURE LOSS FROM CONCRETE
When drying at 100 F and 21% relative humidity after various periods of moist curing or when coated with sealing compound.

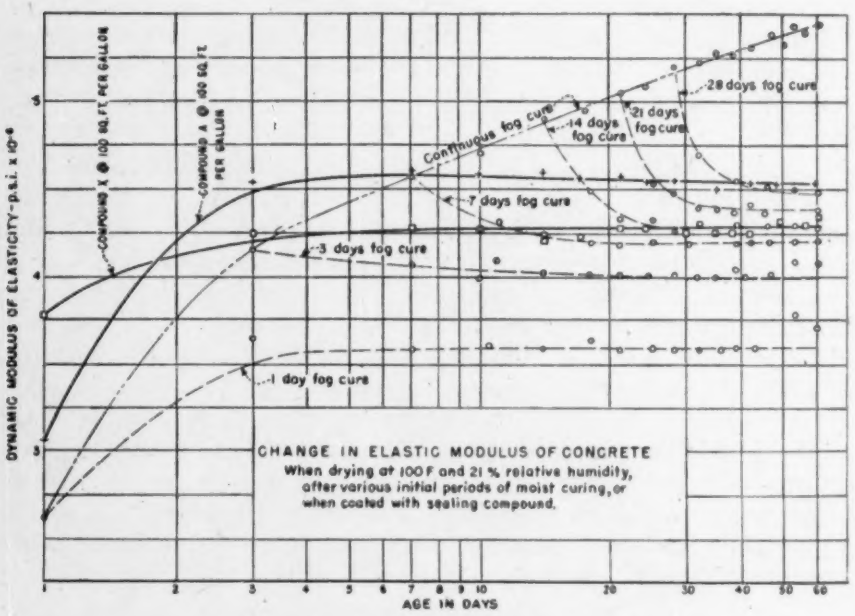

DYNAMIC MODULUS OF ELASTICITY - p.s.i. x 10⁻⁶
AGE IN DAYS
COMPOUND X @ 100 SQ. FT. PER GALLON
COMPOUND A @ 200 SQ. FT. PER GALLON
Continuous fog cure
28 days fog cure
14 days fog cure
21 days fog cure
7 days fog cure
3 days fog cure
1 day fog cure
CHANGE IN ELASTIC MODULUS OF CONCRETE
When drying at 100 F and 21 % relative humidity, after various initial periods of moist curing, or when coated with sealing compound.

Fig. 1 (above)—Moisture loss from concrete
Fig. 2 (below)—Change in elastic modulus

upon them, similar to the moist cured specimens. Such records for two sealing materials, applied at various coverage are also shown upon Fig. 1 and 2. It will be observed that, judging from both figures at sixty days age, the loss of moisture and final elastic modulus of specimens cured with compound X at 100 sq. ft. per gallon compare favorably with the values for the same concrete moist cured for 14 days and then exposed to drying. We may, therefore, conclude that compound X, when applied to formed concrete after it is one day old, at the rate of 100 sq. ft. per gallon, will provide curing equivalent to 14 days moist treatment, under rigorous summer exposure in an arid climate. Similarly, from Fig. 1 we would estimate that compound A, at 200 sq. ft. per gallon coverage, is about equal to 7 days moist curing, and from Fig. 2 that it will give results as good as 28 days moist curing when applied at 100 sq. ft. per gallon.

After sixty days age, the sealing compound is removed from those bars which are coated; all specimens are soaked in water for 48 hours, and then broken in flexure, to determine their modulus of rupture. The remaining pieces are also broken in compression as modified cubes. Thus, considerable information is obtained from a single specimen by which to judge the sufficiency of its curing. Unfortunately, however, the results of the flexure tests are so variable that little correlation appears between modulus of rupture and degree of curing. More consistent results are secured from the compressive strength tests on the modified cubes and their strengths do reflect, to some extent, the amount of curing given to the specimens.

Tests are made at coverages of approximately 100 and 200 sq. ft. per gallon and the results are interpolated to 150 sq. ft. per gallon. The latter coverage has been adopted by the Bureau of Reclamation for use on all structures in the field. After considerable experience with multiple coats and various coverages, it was decided, for simplicity in application and inspection as well as uniformity in procedure, to standardize on one coat of curing compound applied at 150 sq. ft. per gallon. In Table 1 are listed the results secured at this coverage with several typical compounds, offered for use on Bureau work, tested by the above described method. The majority of these materials, excepting the first 5 samples, were found to pass the test for determining moisture-retaining effectiveness, described in detail in the appendix. From Table 1 they may, with the exception of the first 5 samples, be regarded as furnishing the equivalent, or more, of 14 days' moist curing.

REQUIREMENTS OF SEALING COMPOUNDS

It has been found extremely difficult to devise suitable specifications and tests to insure the procurement of satisfactory sealing com-

TABLE 1—EQUIVALENT DAYS OF MOIST CURING

Obtained with varous samples of sealing compound, applied at 150 sq. ft. per gallon coverage, when judged by methods illustrated in Fig. 1 and 2

Sample	Equivalent Days Moist Cure		60 day Compressive Strength psi
	Moisture Loss Basis	Elastic Modulus Basis	
A	9	17	4620
B	13	13	4780
C	13	14	4710
D	13	15	4750
E	14	12	4635
F	15	21	5685
G	15	24	4810
H	17	28	5600
I	20	23	5100
J	22	37	5120
K	28	32	5440
L	31	40	6815
Strength at 60 days for identical specimens fog cured number of days shown		7 days	4760
		14 days	5150
		28 days	5600

pounds. As in many acceptance tests, the time element introduces the necessity of testing in a short period for properties that could be much more accurately determined over several months. The principal requirement of a curing compound is moisture-sealing ability; consequently any specification test should be based primarily on this property, even though it alone may not completely appraise the material. Auxiliary tests may be used to indicate the effect of the compound on the surface hardness and abrasion resistance of the cured concrete; or some indication of the degree of curing may be secured from tests for compressive and flexural strength, although these results must be interpreted with caution since concrete strength is influenced momentarily by its state of wetness or dryness. Other qualities by which to judge a sealing compound are: its application properties; its sprayability and the coverage that can be obtained on a vertical concrete surface without running or sagging; the rapidity with which it dries and avoids tackiness and collecting dirt; the toxic effects of its volatile constituents on workmen when used in poorly ventilated places; the fire hazard with low flash solvents; its bond to the concrete; its durability under sunlight exposure; its light and heat reflectivity; and the general appearance of the concrete after it is applied.

Everything else being equal, the sealing compound that shows the greatest efficiency for retaining moisture in concrete will provide the

best curing on the job. Numerous methods for estimating moisture sealing efficiency in the laboratory have been advanced and the American Society for Testing Materials has issued a tentative standard on this subject under designation C156-44T. The Bureau of Reclamation has been using a method which, while it is not completely satisfactory, has been found to provide results with reasonably close duplication and which has been effective in limiting the selection of sealing compounds to high quality products. This performance test was designed to simulate, as close as possible within the laboratory, actual construction conditions for unformed concrete as regards finishing of the concrete, application of the compound and the exposure conditions existing in semiarid regions during the hot summer.

The details of this test will be found in the appendix. Essentially, observations of moisture loss are made on mortar specimens contained in shallow pans, their tops coated with the subject material, after seven days storage in an atmosphere held at 100 F and 21 per cent relative humidity. A relative humidity of 21 per cent is readily maintained in the tight cabinet holding the specimens, by placing within it a shallow container holding a saturated solution of calcium chloride with excess salt crystals. Bureau specifications require that the test specimens, when coated with the sealing compound under test, at a coverage of 150 sq. ft. per gallon, shall lose no more than 40 grams of moisture. Some typical test results are listed in Table 2 together with the equivalent days of water curing found for the samples by the previously described method.

AVAILABLE MATERIALS

As the result of insistent demand a surprising number of materials have been marketed as sealing compounds, with qualities as varied as

TABLE 2—MOISTURE-RETAINING EFFECTIVENESS OF VARIOUS SEALING COMPOUNDS

As determined by acceptance test (see appendix), compared with the equivalent days of moist curing found for the same samples.

Sample	Moisture Loss in Grams			Equivalent Days Moist Cure*
	Sq. Ft. per Gallon Coverage			
	100	200	150	
1	46.5	62.4	55.0	9
2	15.2	25.2	20.0	12
3	34.2	38.0	36.0	15
4	19.5	21.5	17.6	16
5	11.5	22.5	17.0	21
6	10.0	26.0	18.0	25

(The total water originally contained in the specimens is approximately 600 grams.)
*At a coverage of 150 sq. ft. per gallon.

their number. It is suspected that many of them are attempts to duplicate materials of more secure background. Consequently, indiscriminate selection of sealing compound without testing is dangerous practice and may easily lead to trouble.

Sealing compounds may be classified as: 1) Black (bituminous) compounds—a) Coal-tar cut back, b) Asphalt cut back and c) Asphalt emulsion; 2) Clear compounds—a) Solvent types and b) Emulsions and 3) White-pigmented compound.

Coal-tar cut back, as used for sealing compound, consists of common coal tar, thinned to a sprayable consistency with a coal-tar solvent. Asphalt cut back is prepared similarly with a medium hard asphalt and petroleum solvent. Asphalt emulsion consists of a dispersion of soft asphalt in water. There is some objection to the latter because it remains tacky for too long a period, but it has been used successfully, particularly on tunnel-lining and other unexposed concrete or structures which are backfilled within a few days. It has the advantage in confined spaces of giving off no toxic or odorous fumes as the cut backs do when the solvent evaporates. All black, bituminous compounds are low in cost, under 50 cents per gallon, and they may be readily obtained without necessarily specifying them by trade names.

The proprietary solvent types of clear compound embrace a rather wide field. Some are composed simply of resin and solvent, others combine waxes, drying oil, and water-repellent substances as the solids component, while still others are essentially varnishes. The amount of residue left upon evaporation of the solvent may range from 10 per cent up to 50 per cent of the whole. These compounds are easily sprayable, and most of them can be applied with ordinary hand-operated orchard-type spraying equipment, although most reliably uniform applications are secured with paint spraying equipment, using a pressure pot. Most manufacturers have been able to supply compounds formulated from easily procurable raw materials. The cost of the proprietary clear compounds is usually from $0.50 to $0.80 per gallon, although some may run as high as $1.25.

Although clear emulsions have been marketed at relatively low cost, as a whole they have not been as popular as the solvent type. However, considerable value has been ascribed to a paraffin-linseed-oil emulsion that anyone can make. This compound is referred to as P. J. emulsion, and has been described in detail in the ACI JOURNAL, June, 1942, p. 521. The ingredients of P. J. emulsion are common, readily available raw materials and the cost is lower than for solvent-type clears, being about $0.50 per gallon.

White-pigmented compound may be best described as a solvent-type clear compound to which high-hiding-power white pigment has been

added.[6] It is marketed as a ready-mixed, sprayable compound at a cost of around $0.65 to $1.00 per gallon. A titanium base white pigment is recommended but lithopones are suitable.

THE EFFECT OF A MOIST SUBGRADE

The advantage resulting from placing concrete, such as road slabs and canal lining, on moist subgrades is common knowledge. However, some tests made to evaluate the effect of subgrade moisture are regarded of such interest that their results are presented in this paper.

The tests were conducted in the months of August and September, with exposure to the weather, during a period in which 2.4 inches of rainfall was experienced. Air temperatures ranged from 40 to 93 F. Concrete was placed in 3 x 6 in. cylinder molds resting on soil containing 15 to 16 per cent moisture, which moisture content was maintained throughout the test. Half of the molds were sealed on their bottoms to prevent contact of the concrete with the soil, while the others were open on the bottom so that concrete rested upon the damp soil. The molds remained on the specimens and only the tops of the specimens were exposed to the weather. Some received no curing treatment. Others were cured by covering their tops with wet burlap for 14 days, after which it was removed, and the remainder received coatings of sealing compound, applied two hours after casting. Six different sealing compounds were used, but for the purpose of this report their results are considered as a group.

After 60 days, the cylinders were removed from the molds and sawed in two, normal to their long axis. They were then immersed in water for 3 days and their compressive strengths determined, as shown in Table 3.

The value of moist subgrade in connection with sealing compound is apparent. Although curing appears to be unnecessary if there is a generous supply of moisture from the subgrade, the writers are unwilling to subscribe to such an idea on the basis of these tests alone, because experience has shown that the surface will suffer from cracking and crazing due to lack of early curing treatment despite a well moistened subgrade. The water cured specimens, closed to the moist subgrade, are believed to have benefited from the slight rainfall, whereas such moisture was shed by the cylinders coated with sealing compound. This emphasizes one disadvantage inherent in the use of sealing compounds; while compounds do restrict the loss of moisture from the concrete, they likewise repel beneficial precipitation, until they deteriorate and disappear.

[6] "Canal Lining Cured by Sprayed Coats of White Pigmented Curing Compound," by O. G. Boden, ACI JOURNAL, April 1942, p 449.

TABLE 3—COMPARISON OF EFFECT OF MOISTURE IN SUBGRADE ON STRENGTH OF CONCRETE UNDER DIFFERENT CURING METHODS.

Type of Curing	Placed on Moist Subgrade	Compressive Strength—psi		
		Top Half	Bottom Half	Average Top and Bottom
Sealing Compound 100 Sq. ft. per gal.....	No	4870	5110	4990
14-day wet burlap.....	No	5570	5770	5670
None.....	No	3180	3210	3200
Sealing Compound 100 Sq. ft. per gal.....	Yes	5570	5600	5590
14-day wet burlap.....	Yes	5660	5410	5540
None.....	Yes	5420	6060	5740

The concrete which was given no curing whatever received great benefit from the moist soil. R. B. Gage[7] has previously observed that pavements which were cured only a few hours, but which rested on moist subgrade, had the desired strength and durability, as attested by cores taken from them. However, concrete which had heavy building paper interposed between it and the subgrade, even though receiving water curing, developed only 77 per cent of that placed directly on the subgrade (tested at 11 years age), according to investigations reported by Stanton[8].

These results serve to illustrate that factors, other than restriction of moisture loss from the exposed surfaces, must be considered in some curing problems. It is important to properly evaluate sealing compounds and to require the use of the best materials when using this method of curing, but it is equally important that we understand the part played by other factors which may affect the quality of the concrete.

TEMPERATURE OF CONCRETE AS AFFECTED BY SEALING COMPOUNDS

While the suitability of a sealing compound, as previously stated, depends primarily on satisfactory moisture retention characteristics, there are two further requirements that are also considered important. First, where it is desired to employ membrane curing on concrete structures exposed to direct sunlight, the compound coating should exhibit satisfactory heat-reflecting properties. Second, where compounds are

[7]Discussion of "Tests of Concrete Curing Materials" by F. H. Jackson and W. F. Kellerman, ACI *Proceedings*, Vol. 35, p 500-7.
[8]See previous reference (4).

used on finished structures exposed to visual observation, the finished concrete must present a satisfactory appearance.

Except for white-pigmented compounds, sealing compounds give the concrete a darker color than the natural grey obtained when cured with water. This results in higher temperatures being induced in the concrete from radiant sunlight. Surface temperatures as high as 145 F have been noted in the field on concrete coated with black bituminous sealing material. For this reason the Metropolitan Water District required that black sealing compound be covered with one or two coats of whitewash before exposure to the sun during the heat of the day.[9]

Table 4 shows the temperatures, over a 24-hour period, of concrete coated with typical sealing compounds exposed to the sun's rays during a summer day. These are compared to the temperatures of concrete previously water cured, but exposed in a dry state to the sun. There is no question in the minds of the authors that, during a period of moist curing, when such curing is properly executed, far lower temperatures and less temperature fluctuation will be found. In this respect, water curing cannot be equaled by sealing compounds. However, it will be noted that the temperatures are more satisfactory on concrete cured with white-pigmented sealing compounds than on uncoated concrete after moist curing has ceased.

Most clear compounds have a light-fading dye added to them in order to permit the applicator to readily see the compound on moist concrete, to aid in obtaining a uniform application and to distinguish between coats of the material. Even though these dyes fade to a great extent,

[9] "Solving the Problem of Concrete Curing on the Colorado River Aqueduct" by L. H. Tuthill, *Western Construction News*, August 1935, p 220.

TABLE 4—TEMPERATURES, 24 HOUR PERIOD, OF CONCRETE COATED WITH TYPICAL SEALING COMPOUNDS EXPOSED TO SUN ON SUMMER DAY.

Observation	Black compound	Clear compound	White-pigmented compound	Water-cured
Maximum temperature F				
Surface	134	126	103	117
3-inch depth	119	116	99	107
Air (shade)	87	87	87	87
Minimum temperature F				
Surface	65	65	65	64
3-inch depth	68	66	67	66
Air (night)	62	62	62	62
24-hour range F				
Surface	69	61	38	53
3-inch depth	51	50	32	41
Air	25	25	25	25

they do, together with the waxy or resinous ingredients of the compounds, darken the concrete more than ordinarily desired. Very often, also, they leave the concrete with a blotchy appearance. In an effort to overcome these objectionable characteristics, the Bureau of Reclamation experimented with additions of white pigment to clear sealing materials. The result has been the adoption of white-pigmented sealing compound, which gives the concrete a pleasing appearance for several months, after which it gradually, but uniformly, weathers away. Uniform coverage at the rate specified can be determined at a glance, consequently it has been more easily obtained with this sealing compound than with others and inspection of the curing operation has been much simplified. Since the white surface resulting with this compound materially lowers the temperature of concrete exposed to the sun, Bureau of Reclamation concrete specifications no longer require that the fresh concrete be shaded for three days when this type of sealing compound is used.

FIELD EXPERIENCES

Sealing compounds were first introduced for use on concrete work located in regions where water was scarce. The difficulties and expense involved in providing effective water curing, particularly for canal lining concrete in desert areas often made the cost of this method of curing prohibitive. Once introduced, the contractors were quick to grasp the utility of compound curing as a means of simplifying and speeding their work, frequently at lower cost, even where water for curing was abundant. The engineer has not been reluctant to accept this alternate method because, after satisfying himself that sealing compounds would properly cure the concrete, it meant less argument over the sufficiency of the curing operation. Such considerations still hold and furnish an incentive for the use of sealing compounds today.

Except under the most severe drying conditions, as for example during the summer months in the desert regions of the southwest, it is believed that sealing compounds can be used to obtain the desired degree of curing. They are most effective in curing slab work, such as pavement and canal lining, where additional moisture is available from the subgrade, and they are least effective for thin sections and structural members having a high ratio of exposed area to volume.

From examinations of much concrete canal lining, cured by various methods, it has been noted that crazing of the surface concrete has been reduced, or almost completely eliminated, when sealing compounds of the clear or white-pigmented type have been used. That the surface condition in this respect is improved over water or wet-burlap cured concrete is certain. Concrete cured with black compounds, which were not pro-

tected by whitewash, is on the other hand found to be cracked and crazed more than similar concrete cured with water.

When given a radiant heat reflective coating of whitewash, black bituminous sealing compounds have been found to give satisfactory results. The whitewash, however, weathers away in a nonuniform fashion, leaving the black painted concrete mottled and unsightly. Such objections do not preclude its use on concrete which is later to be backfilled. Where the concrete is not exposed to sunlight, as tunnel lining, bituminous sealing compound without the benefit of whitewash has been found to do a creditable job of curing.

At one time it was the policy of the Bureau of Reclamation to evaluate the curing efficiency of any sealing compounds proposed for use on its work by the contractors. Depending on their moisture retention ability, and general suitability in other respects, directions in regard to the number of coats and coverage for each coat would then be issued to the contractor for each material. This procedure was designed to permit the use of all compounds whose general characteristics were suitable, and simply required that a heavier application of a less efficient compound be used in order to secure a standard amount of curing. This standard has always been the equivalent of fourteen days moist curing. Such procedure was found to be extremely involved and entailed a prodigious amount of testing with much tedious inspection of the curing operations. When multiple coats were required, it was difficult for the inspectors to determine if instructions had been followed. The light-fading dyes incorporated in the sealing compounds to distinguish between coats were not entirely satisfactory and without their use it was sometimes difficult to determine where the clear type of compound had been applied to moist concrete.

As a result of such experiences the Bureau determined to limit sealing compounds to those which would furnish the desired curing when applied in a single coat at a coverage of 150 sq. ft. per gallon. The decision was further made to use only white-pigmented compound, whose virtues have already been discussed, and whose uniform application is achieved almost automatically when the density and uniformity of its color and hiding power are used as a guide. When applying compound to rough concrete surfaces, consideration must be given to the larger surface area presented within given dimensions and accordingly heavier applications should be made.

It has been found that, if sealing compound is applied to a dry concrete surface, much of it may "strike into" the surface and, unless an excess of compound is used, a continuous membranous film will not be secured. Its application to slab concrete should immediately follow the final finishing. If such immediate application is delayed, the concrete

surface should be kept moist by lightly spraying with water until the compound is applied. Formed concrete should be sprayed with water after the forms are stripped and before sealing compound is applied.

There are some who advocate a little water curing in advance of the use of sealing compounds. Often, in cases where dissatisfaction has been expressed over the results of compound curing, preliminary water curing of from several hours up to 48 hours will remedy the trouble. As an example, considerable difficulty was experienced on one job from checks appearing in the concrete placed as lining on the side slopes of a canal. This concrete was being cured with sealing compound and, although the checks could be closed by troweling, they would reappear shortly after application of the compound. By curing the concrete for 24 hours with wet burlap prior to compound application, the checking was eliminated. From an economical standpoint it would seem desirable to avoid recommending preliminary water curing as an indispensable step in the use of sealing compounds, however, it remains as a valuable adjunct to membrane curing as the exigencies of the situation demand.

Paint spraying equipment, using a pressure-pot with an agitator, has been found most desirable for applying sealing compound. A gun with extension handle permits the operator to cover a wide area and reach otherwise inaccessible concrete surfaces. While hand spray outfits, designed to distribute insecticides in orchards, have been used to apply some of the lighter clear compounds, they require conscientious effort on the part of the operator and will not handle white-pigmented material.

Curing of concrete by means of sealing compounds has been found very expeditious and convenient in certain cases. As an alternate to water curing it avoids the undesirable sloppy conditions usually accompanying the latter method. In confined quarters, such as tunnel lining operations, better working conditions result. Where compacted backfill is placed adjacent to the concrete, it can proceed without interruption or interference with moisture control on the compacted earth.

SUMMARY

The authors have attempted to present information necessary for the efficient use of sealing compounds as curing agents for concrete. Most of this information has come from the experiences on concrete work performed for the Bureau of Reclamation on its projects in the arid west. The climate in this region requires extra care to insure that concrete receives sufficient curing and it has been the practice to require 14 days moist cure, or its equivalent, for concrete in this area. It is believed that any sealing compound accepted for such work will likewise do a creditable curing job in any other part of the country. The conditions under which sealing compounds are permitted to be used and instructions

concerning them on Bureau work are to be found in the attached appendix.

In summarizing, there are listed below, the advantages and disadvantages of curing with sealing compounds. Most of the points have been discussed in the body of the paper, although a few additional ones appear without previous mention.

Advantages

1. Requires less inspection. Once the surface is sealed, no further attention is required, except for occasional repairs of damage to the coatings.

2. Creates minimum interference with other construction activities. Frequently, water curing operations make it difficult or impossible to carry on some features of the job until the curing period is finished.

3. Eliminates sloppy, disagreeable working conditions in enclosed areas, for example, in tunnels, siphons, and conduits.

4. Necessary equipment is much less than for water curing on the larger or more extended work.

5. Prolongs curing beyond the usual practicable time for water curing. An accurate appraisal of this feature is needed; it must be borne in mind that this may revert to a disadvantage if for an extended period the coating excludes moisture otherwise available from rainfall and condensation.

6. Prevents sudden dryout at an early age as at the end of water curing operations. This probably accounts for the fact that crazing in compound-cured concrete is a rarity if appreciable moisture is not lost prior to application and heat absorption is prevented.

7. Makes it possible to do a good job of curing at less expense than water curing. This is particularly true in regions where water must be transported great distances.

8. The use of white-pigmented sealing compound lowers the temperature of concrete exposed to sunlight, reduces the fluctuating temperature range of such concrete and presents a desirable appearance. It is not superior to well conducted moist curing in this respect, until after moist curing ceases.

Disadvantages

1. Although the length of inspection period (time required to be devoted by inspectors) is less than for water curing, greater attention to detail is necessary. The application of sealing compound must be correctly timed so that the concrete is in the most advantageous condition with respect to saturation when the compound is applied. It is

necessary to compute coverage rates, to see that the specified amount is applied uniformly to all areas.

2. Sealing compounds require careful inspection and considerable acceptance testing to insure the procurement of a reliable compound.

3. Serious damage to the surface quality of concrete will result from breakdown or delay in application where there is no standby equipment for alternate water cure.

4. Reasonable protection against damage to coatings and repair of damaged areas is required.

5. More or less discoloration of concrete surfaces results, depending on the type of compound. Even the clear compounds mar the natural appearance of concrete to some extent, at least until the coating weathers away.

6. Except for white-pigmented compounds, sealing compounds create a darker colored surface than water curing and as a result induce higher concrete temperatures under sunlight exposure. Corrective measures, such as whitewash over black coatings, usually involves unsightly appearance for long periods.

7. Depending on the compound, the concrete is deprived of benefit from precipitation and condensed moisture until the compound weathers away.

8. Some sealing compounds contain low flash solvents which require care in handling. The fumes from these may also be toxic and present some physiological hazard when used in poorly ventilated spaces.

9. The effectiveness of sealing compounds is questionable for curing thin sections or structural concrete.

CONCLUSIONS

Analysis of the information presented here and of that from other sources leads to the following conclusions.

1. Sealing compounds are available which, if handled properly, can be used under appropriate conditions to secure effective and economical curing.

2. Other factors, such as moisture in subgrade, moisture condition of the concrete when coated, and subsequent temperatures, have as much or more bearing on the effectiveness of the curing as the moisture-sealing properties of acceptable compounds.

3. Preliminary water curing is a valuable adjunct to compound curing. Although not prescribed for all circumstances, its use is invariably beneficial and is to be encouraged, especially where the climate imposes severe drying conditions.

4. The use of white-pigmented compound, although somewhat more expensive, is one of the best solutions to the problem of controlling temperature rise in concrete exposed to the sun. This type of sealing compound presents a pleasing appearance except for a short period prior to its final disappearance, and, due to its color, is readily applied at a uniform rate to the desired coverage.

5. Selection of satisfactory sealing compounds requires the use of competent specifications and testing procedures.

APPENDIX

A SPECIFICATION FOR WHITE-PIGMENTED SEALING COMPOUND FOR CURING CONCRETE

1. Materials and manufacture.

Sealing compound shall consist of finely ground, white pigment and vehicle, ready-mixed for immediate use without alteration other than stirring. The compound shall be such that it can be used with safety under properly controlled conditions and shall conform with the requirements of these specifications when applied in one coat at a coverage of 150 square feet per gallon (hereinafter referred to as the specified coverage).

2. Detail requirements.

Sealing compound shall comply with the following physical requirements:

(*a*) *Adhesion.* A coating of sealing compound, sprayed on a moist concrete surface, either prior to or subsequent to the time at which the concrete has attained its initial set, shall adhere firmly to the concrete surface after the concrete is hardened.

(*b*) *Color.* The compound shall provide a uniformly white appearance and shall effectively obscure the original color of concrete when applied at the specified coverage. When applied at a coverage rate of 100 square feet per gallon on the checkerboard hiding-power chart described in Federal specification TT-P-56, amendment 1, the coating shall obscure the black squares. When applied to concrete, sealing compound shall not darken or yellow appreciably under outside exposure.

(*c*) *Drying properties.* When applied on a damp vertical concrete surface, in one coat at the specified coverage, and exposed at an atmospheric temperature of 100 F and relative humidity of 21 per cent, the sealing compound shall be dry to touch within four hours. The compound, when so applied and exposed, shall neither run nor sag on the concrete surface.

(*d*) *Viscosity.* The viscosity of the compound at a temperature of 77 F shall not exceed 60 K. U. as determined by method 428.1 Federal specification TT-P-141a.

(*e*) *Weathering.* The compound shall be of such composition that the coating formed by its application on concrete surfaces will provide an effective seal for at least 28 days.

(*f*) *Reaction between sealing compound and concrete.* There shall be no evidence that new concrete surfaces are softened by reaction with the sealing compound.

(*g*) *Performance of sealing compound.* The performance of the coating of sealing compound shall be measured in terms of its moisture-retaining characteristics. The moisture loss, when determined in accordance with the method described in paragraph 3, shall not exceed 40 grams.

3. Test method for determining moisture-retaining effectiveness of sealing compounds.

(*a*) *Apparatus—Molds.* The molds shall be rectangular pans made of metal, glass, hard rubber, or suitable plastic material and shall be water-tight. They shall be con-

structed rigidly to prevent distortion and shall be cleaned thoroughly before each use. The molds shall have the following inside dimensions: at top, 6 in. x 12 in.; at bottom, 5¾ in. x 11¾ in.; depth, 2⅛ in.

(*b*) *Test specimens—Mortar:* Mortar for test specimens shall consist of portland cement, air-entraining agent, sand, and water, all well mixed in the proportions, by weight, of 1 part of cement, 0.003 parts neutralized Vinsol resin (NVX), 2.2 parts of sand, and 0.4 parts of water. The portland cement shall conform to the Standard Specifications for Portland Cement, A.S.T.M. Designation: C 150-44, type II. The neutralized Vinsol resin shall be first dissolved in the mixing water. Sand for mortar shall be natural silica sand from Ottawa, Illinois, and shall be graded as follows:

U. S. standard sieve	Per cent retained
No. 100	98 ± 2
No. 50	72 ± 5
No. 30	2 ± 2
No. 10	None

Mixing: Mixing shall preferably be done in a room in which the air temperature is 70 (± 5) F and the relative humidity 40 to 60 per cent. The temperature of the mortar at the time of molding shall be 70 (± 5) F. The cement and sand shall be mixed dry in a nonabsorptive vessel and the specified quantity of water shall then be added. The dry mixture shall be permitted to absorb the water for one minute. Mixing shall be accomplished by suitable mortar-mixing machine, or by continuous kneading and squeezing with the hands for two minutes. Rubber gloves shall be worn if hand mixing is employed.

Molding: The mold shall be filled with mortar and the mortar puddled with the gloved fingers only to the extent required for uniform and thorough consolidation. Excess mortar shall be removed and the surface smoothed with the gloved hands, immediately after which the surface shall be finished with one lengthwise pass of a wooden float using firm pressure.

Storage: Immediately after molding, the molds containing the specimens shall be placed in the curing cabinet in an atmosphere maintained at 100 (± 2) F and a relative humidity of 21 (± 2) per cent. The specimen shall not be subject to vibration and shall be level. Air shall be circulated in a horizontal plane in the cabinet at a velocity of 160 (± 30) feet per minute. Four specimens shall be prepared for each sealing compound to be tested.

Application of sealing compound: The specimens shall be removed from the curing cabinet as soon as the surface water has disappeared and it is possible to brush the surface lightly without smearing or drawing more water to the surface. Immediately upon removal of a specimen from the curing cabinet, the junction of the specimen and the mold shall be sealed by means of a sealing material composed of a mixture of three parts of Bexin resin and one part of Vinsol resin. This sealing material shall be heated to a fluid consistency and applied to the edges by means of a small brush, and shall not extend more than one-quarter inch onto the top surface of the specimen. The mold containing the specimen shall then be weighed to the nearest 0.5 gm. A single coat of the sealing compound to be tested shall be sprayed on each of four test specimens. Approximately one-half of the thickness of the coating shall be applied by moving the spray gun back and forth in one direction and the remainder of the coating shall be applied immediately thereafter by moving the gun at right angles to the direction of the first application. Two specimens shall be coated uniformly at an approximate coverage of 100 sq. ft. per gal. and two at 200. The actual coverage shall be determined by weighing the spray gun to the nearest 0.1 gm. before and after application of the compound to each pair of specimens, and correcting the difference in weights for overspray, as here-

inafter described. A backstop of heavy craft paper shall be used to catch the overspray. The backstop shall be weighed accurately to the nearest 0.1 gm. before spraying the specimens, and again after the overspray on it has dried.

Solids content: The solids content of the sealing compound shall be determined in accordance with paragraphs 7 and 8 of A.S.T.M. Designation: D 154-43.

Computation of actual coverage of test specimens: From the weight of solids deposited from the overspray on the backstop during the coating operation, and from the solids content of the sealing compound, a correction shall be calculated and added to the weight of the spray gun and its contents after the coating operation. The weight per gallon of the compound, in grams, shall be divided by the difference between the corrected weight of the spray gun after spraying and the weight of the spray gun and its contents before spraying, and the quotient multiplied by the coated area in square feet, to obtain the actual coverage in square feet per gallon.

(c) *Exposure of specimens and determination of moisture loss—Exposure:* Immediately after application of the sealing compound, the specimens shall be placed in the curing cabinet and maintained at a temperature of 100 F and a relative humidity of 21 per cent.

Determination of moisture loss: After seven days storage in the cabinet, the molds containing the specimens shall be weighed to the nearest 0.5 gm., and the loss in weight based on the combined weight of mold and specimen immediately before the application of the sealing compound, shall, after correction for the weight of solids in the sealing compound applied, be considered as the moisture loss from the specimen. The moisture loss corresponding to the specified coverage of 150 square feet per gallon shall be determined by interpolation from the average losses measured for the two test coverages.

INSTRUCTIONS FOR THE PROPER USE OF SEALING COMPOUNDS

1. Preparation for use.

Sealing compound, as delivered on the job, shall be ready-mixed. At the time of use the sealing compound shall be in a thoroughly stirred condition, such that the pigment is uniformly dispersed throughout the vehicle. Stirring shall be accomplished by an effective power-operated mechanical stirrer. Sealing compound shall not be diluted by addition of solvents or thinners or be altered in any manner without specific approval. Sealing compound that has become chilled to such an extent that it is too viscous for satisfactory stirring or application shall be heated until it has proper fluidity, but shall not be heated to a temperature higher than 100 F. Heating shall be accomplished by indirect application of steam or hot water or by other approved methods. Heating of sealing compounds by direct exposure of the container to fire will not be permitted.

2. Application.

Sealing compound shall be applied to the concrete surfaces by spraying and shall be applied in one coat at a coverage of 150 square feet per gallon. Spraying equipment shall be of the pressure-tank type, such as the De Vilbiss Portable Spray-Painting Outfit NJE-620 equipped with feed paint tank agitator QS-603, which provides for continual agitation of sealing compound during coating operations. Ordinary orchard-type hand-spray outfits shall not be used. In order to facilitate thorough and complete coverage of the concrete surfaces to a uniform white color, approximately one-half of the required thickness of coating shall be applied by moving the spray gun back and forth in one direction and the remainder of the coating shall be applied immediately thereafter by moving the gun at right angles to the direction of the first application.

When sealing compound is used on an unformed concrete surface, application of the compound shall commence immediately after finishing operations are completed; *provided*, that in the event application of the compound is delayed, the concrete surface shall be kept continually moist until the compound is applied. When sealing compound is to be used on a formed concrete surface, the surface shall be moistened with a light spray of water immediately after the forms are removed and shall be kept wet prior to application of the compound. It is of utmost importance that the concrete be in a thoroughly water-saturated state at the time of application of the sealing compound. Spraying of the compound, therefore, shall not commence until the concrete has been soaked to such an extent that the surface will not readily absorb more moisture. As soon as the surface film of moisture disappears and there is an approach to surface dryness, the sealing compound shall be applied; *provided*, that in the event application of the sealing compound is delayed, the specified moistening of the concrete surface shall be continued until the sealing compound is applied. Special care shall be taken to insure ample coverage with the compound at edges, corners and rough spots of formed surfaces. After application of the sealing compound has been completed and the coating is dry to touch, any required patching of the concrete surface shall be performed. Each patch, after being finished, shall be moistened and coated with sealing compound in accordance with the foregoing instructions. Traffic and other operations shall be so handled as to avoid damage to coatings of sealing compound for a period of not less than 28 days. Where it would be impossible to avoid damage, as in the case of flat surfaces, which, within 28 days after application of sealing compound, must be subjected repeatedly to foot or vehicular traffic or handling of materials, the sealing membrane shall be protected by a covering of sand or earth not less than one inch in thickness or by other suitable and effective means. The protective covering shall not be placed until the sealing membrane is completely dry, and not sooner than 24 hours after application of the sealing compound. Any damage to the sealing membrane shall be repaired without delay.

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Discussion of a paper by R. F. Blanks, H. S. Meissner and L. H. Tuthill:

Curing Concrete With Sealing Compounds*

By GEORGE W. WHITESIDES, P. G. WHITMAN and AUTHORS

By GEORGE W. WHITESIDES†

Despite rather extensive studies before the war, membrane curing compounds have suffered rather badly through their general adoption in a period that made accurate evaluation difficult through a shortage of testing facilities and personnel. However, the paper by Messrs. Blanks, Meissner and Tuthill carries exceptional weight since it is based on extensive studies in an area requiring a more than normal need for curing and on laboratory data thoroughly carried out.

The method chosen for the evaluation of laboratory specimens is interesting and undoubtedly deserves further study. I ask you to note that membrane cured specimens are listed in equivalent fog room time for the same moisture loss and elastic modulus. This is, I think, unique but should yield evidence of greater value than the more usual method of evaluation against specific periods of fog or wet curing.

I suggest, however, that particular care be exercised in any contemplated laboratory studies intended to compare data obtained on membrane cured specimens whose surface-mass relationship is greatly different than that occurring in field practice with similar specimens fog cured—again a curing method usually different from that actually obtained in field practice.

Several points, however, call for specific comment. Our experience leads us to believe that temperature change and thermal shock are even more favorable to membrane curing than might be implied from the data presented. Temperature changes slowly in membrane cured concrete with the possible exception of the cooling effect of sudden showers. The application of curing water as generally done is usually intermittent, resulting in frequent and sudden thermal changes and stresses particularly detrimental in the early stages.

*ACI Jl., Apr. 1946; *Proc.* v 42, p. 493
†President, George W. Whitesides Co., Inc., Louisville, Ky.

The evidence we have does not lead us to believe that pigmented compounds are necessary in the country east of the Mississippi but this is based entirely on observation and cannot be supported with data. At any rate we feel it obvious that the heat pickup of either the clear or pigmented compounds does not approach the heat pickup of concrete under curing paper.

A second point deserving comment is the need to clarify the conclusion given on the necessity or even desirability of preliminary water curing. The paper notes that the crazing of surface concrete has been reduced or almost completely eliminated when sealing compounds of the clear or white pigmented type have been used. Further, "that the surface condition in this respect is improved over water or wet burlap cured concrete is certain."

This statement seems to be in disagreement with the comment that sometimes where dissatisfaction has been expressed over the results of compound curing, preliminary curing of from several hours up to 48 hours will remedy the trouble. We can confirm his findings that membrane curing compounds with a high degree of water retention invariably eliminate crazing and cracking. Preliminary water curing, however, in our opinion, unless it is more carefully done than is normal in field practice leaves the pores unsaturated and makes difficult the application of continuous membrane film without excessive absorption.

Frankly, we have always thought that a great advantage in membrane curing lies in: 1) the possibility of earlier application; 2) the elimination of intermittent curing. Please remember that membrane curing is independent of wind currents and burlap mats can be displaced and quite often are, especially on vertical surfaces.

The authors emphasize a widely held objection to membrane curing: that the usual methods of application are inadequate. I suggest that specifications be drawn to eliminate this since spray apparatus should and can be designed if the manufacturers are given the necessary stimulus, to give a degree of accuracy of application within very close limits. This same comment is applicable to his remarks on discoloration since materials of high efficiency are available that are very satisfactory in this respect.

Little criticism of the specifications current in the Bureau of Reclamation can be offered. They apparently recognize the fact that membrane curing must be evaluated, everything else being equal, primarily on the basis of water retention. Unlike the A.S.T.M. method, the Bureau recognizes that membranes can be applied earlier in the construction period than can wet curing. Moisture loss, therefore, prior to the time that burlap can be used, should not be made a part of the tests on membrane curing.

There is, as most of us know, a considerable difference of opinion as to the type of panel used, and I note that the Bureau reports moisture loss in weight rather than percentage. Since the surface area is given, the unit loss per unit area can be arrived at. I think it desirable, until such time as all testing methods agree on a particular type specimen, to evaluate membrane curing on weight loss per unit area, since by so doing tests can be corrolated.

As a manufacturer of membrane curing compounds, I urge the need: 1) to finally evaluate the method itself, and 2) to promulgate satisfactory specifications, uniform as far as conditions allow, and particularly uniform as to test methods. If an objection to the use of membrane curing compounds is found in the present methods of application, the inclusion in the specification of a desired accuracy will result in the development of methods to eliminate that objection. This is equally true of discoloration, harmful reactions and other objections which are perfectly valid. For most of the work on membrane curing compounds in recent years, as far as the manufacturers are concerned, has apparently been devoted largely to cost reductions. This is undesirable and unnecessary since the method in itself contains obvious economic advantages. The present disorder can only discourage the continued satisfactory use.

By P. G. WHITMAN*

I consider the paper as a whole a great addition to the available information on the membrane method of curing concrete, and wish to commend the authors for the very able and thorough presentation of the subject.

It seems to me that certain of the comments or comparisons made by the authors are based on the ultimate strength which any concrete could attain under perfect or ideal conditions, whereas for the general consideration of membrane curing it might better be from the angle of practical strength, or the highest strength obtainable from good construction procedure.

I concur very strongly with all that is said in "Factors in proper curing", "Requirements of sealing compounds", and "Available materials". These subjects have been covered in an excellent manner.

I am wondering whether there is a typographical error in Table 2, insofar as "Sample 4" is concerned. I note that all other moisture loss figures for the 150 sq. ft. per gal. coverage are the mean between the 100 and 200 sq. ft. coverage. For Sample 4 the 150 sq. ft. figure is 17.6 gm., which is almost 2 gm. less than the 19.5 gm. shown for the 100 sq. ft. coverage.

*Hunt Process Co., Los Angeles 1, Calif.

This might well be the place for mentioning my belief that it would be better in determining the sealing quality at the 150 sq. ft. coverage, to actually test it at that rate, and not assume that the sealing quality is the mean between the results obtained from specimens coated at 100 and 200 sq. ft. per gal. I am quite positive in my belief that the actual sealing quality of the cure at 150 sq. ft. is better than the mean between the two other rates of coverage. Consequently, it is my thought that the test procedure might be changed in this respect.

I agree generally with the comments made under the heading "The Effect of a Moist Subgrade". I can not help wondering whether the results from six different sealing compounds were so close together that none of them showed any appreciable difference from the others. I am also wondering whether the size of the cylinders was so small that the results were not truly applicable to actual construction. Up until four or five years ago when membrane curing became well established, many tests were performed to endeavor to obtain comparison between membrane and water curing. In a great many cases this consisted of making the standard 6 x 12-in. cylinders, curing half by the standard water curing, and their companions by removing the cylinders from the mold and completely coating with a popular make of curing compound. The cured cylinders were left exposed out-of-doors near some job or laboratory. I have records of a large number of these tests where the cured cylinders showed at least equal strength with the water cured, and some up to 120 percent of the water cure. In each case, as well as I can remember, no effort was made to have these cylinders rest on moist ground.

There is no question in my mind that the pigmented cure does a very effective job insofar as controlling the temperature of the concrete is concerned.

There is only one point in the authors' comments under the heading "Field experiences" on which I have any comments to make other than commendation. This is regarding statements made to the effect that checking was possibly eliminated on a certain project by the preliminary curing of the concrete with wet burlap prior to the membrane cure application. As membrane curing can be started before water curing can be instituted it seems to me there must have been other reasons for the lack of checking rather than the 24 hours of water curing. It has been my experience wherever checking, due to evaporation of the mixing water, has apparently occurred in concrete cured with a good curing compound, that incipient checking was already present before the compound was applied. I know of numerous cases where membrane curing has stopped checking, in many instances after water curing had failed in that respect.

I concur entirely with the advantages stated, but am wondering whether the fact that a membrane cured surface does not pick up moisture from rainfall or condensation is actually a disadvantage. If membrane curing retains sufficient of the mixing water in the concrete to do a perfectly satisfactory job, then it does not seem logical that it is a disadvantage if the concrete can not pick up additional moisture. Even on concrete which is not subjected to abrasion, such as traffic, the film from the curing compound disintegrates sufficiently within some months so that the concrete can absorb moisture. Consequently, the benefits derived from this source would continue from that point on.

Under "Disadvantages" I feel the statement made in 9 called "The effectiveness of sealing compounds is questionable for curing thin sections or structural concrete", is too strongly worded. I am presuming in this respect that "thin sections" refers to concrete not placed on a moist subgrade. I could repeat here the comments made above mentioning the results of the crushing tests using the cylinders. In considering the ratio of surface area to mass of a standard cylinder exposed on all sides except the base, it corresponds with a slab of concrete slightly over 1-in. thick exposed on all sides. If tests indicate that under such conditions concrete has equal strength with laboratory water curing, it would appear that it should be satisfactory for curing of all types of concrete where the relationship between mass and surface area are much more favorable. Also no consideration, in my opinion, has been given to the difficulties of properly water curing this type of concrete construction.

I can not agree with the authors' conclusions that "Preliminary water curing is a valuable adjunct to compound curing". This procedure has not been the general practice for a number of years, and is unnecessary if the finishing operations are not unduly delayed and the curing compound applied as quickly as possible thereafter. I will agree that where the application of the curing compound is held up unnecessarily long after the finishing or removal of the forms that the concrete should be thoroughly wetted before the application of the curing compound.

AUTHORS' CLOSURE

Mr. Whitman reveals the detailed amount of study he has given the paper, by calling attention to data which is in error. The figures of 19.5 and 17.6 gm. moisture loss, for Sample 4 in Table 2, are transposed and should be rearranged under the columns "150 sq. ft. per gal." and "100 sq. ft. per gal.", respectively.

In the introductory remarks under the title, "Factors in proper curing," the authors discussed all phases of curing in a general manner, including what might be termed ideal curing, or the ultimate that could be achieved.

References to this desirable state have concerned Mr. Whitman, who would prefer to see comparisons made between curing accomplished with sealing compounds and the practical amount of curing that is actually accomplished without them. It is admitted that, even under acceptable practice, the amount of curing ordinarily given concrete by the application of water fails to approach the ideal. The latter is beyond what can, ordinarily and reasonably, be secured. As brought out in the paper, curing with sealing compounds can be as effective as some acceptable amount of moist curing, which falls short of the ideal or ultimate.

Both Mr. Whitesides and Mr. Whitman question the desirability or need for some preliminary moist curing of the concrete prior to application of sealing compound. The authors do not recommend this as a regular procedure, except for an initial wetting of formed surfaces after forms are stripped. However, its effect on horizontal finished surfaces can only be beneficial, and in cases where difficulties with crazing are encountered, such an expedient is suggested for trial. There is no disagreement between the authors and Mr. Whitman over the necessity for preliminary moist curing, should the application of sealing compound be delayed.

The statement that membrane curing can prevent the concrete from receiving the benefit of rainfall has not met with complete approval of the discussors. Perhaps it should have been made clear that a comparison was being made between moist cured concrete and membrane cured concrete, both of which later receive precipitated moisture. Rainfall which occurs after water curing has ceased, will augment and extend such water curing. It will not benefit the membrane cured concrete in the same fashion, because the sealing material will shed this beneficial precipitation. It matters not that both concretes may have been cured to an equal degree up to the point when rain descends; the final effective curing received by the moist cured concrete will be greater, while that of the sealed concrete will not be similarly enhanced. This disadvantage should not, however, be regarded as serious. The authors have qualified this as existant only in the interim before the sealing compound weathers away.

Mr. Whitman feels that the statement under disadvantage 9 is too strongly worded. This statement questions the effectiveness of sealing compounds for curing thin sections, or structural concrete. As the test data given on 3- x 3- x 15-in. bars may be considered to be thin sections, or representative of very slender structural concrete, his criticism is justified. The paper shows that concrete in these test specimens may be cured by means of sealing compound to some equivalent degree of moist curing. When stating disadvantage 9, the authors had in mind

the greater difficulty of curing structural concrete as compared to concrete resting upon a moist subgrade. Reconsidering this point, we would restate disadvantage 9 as follows, "Sealing compounds provide less effective curing for thin sections or structural concrete than when used to cure slabs on a moist subgrade."

Mr. Whitesides correctly points out that one advantage of membrane curing lies in the elimination of intermittent curing, often a fault in moist curing procedure. The authors were loath to discuss the laxities often encountered in water curing and, when comparing the various methods of curing, they preferred to presume that all were properly performed. Continuous water curing can be secured by insisting upon it, and similarly there are important details concerning membrane curing which require an inspector's attention. However, in this respect the advantage of curing with sealing compounds resides in the fact that the period of inspection is shorter. Once the sealing material is properly applied, the inspector is no longer required to exercise the constant vigilance that several days of water curing may demand of him.

To facilitate selective distribution, **separate prints** of this title (42-19) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Aug. 15, 1946.

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Radiant Heating by Reinforced Concrete*

By JOHN R. NICHOLS†
Member American Concrete Institute

SYNOPSIS

The question is examined whether the embedment of warm water pipes in structural concrete for the purpose of "radiant heating" (or "panel heating" as it is sometimes called) would endanger the integrity of the concrete. Some evidence is adduced in support of the tentative view that with reasonable restrictions on the installation, the results would not be too bad.

Radiant heating is the system of heating rooms or spaces by warming the walls, floor and ceiling, one or more, or a part of them. It differs from other methods of heating principally in that the immediate source of heat to the room is of large area and moderate temperature instead of small area and high temperature. The orthodox sources of heat with which we are most familiar are the fire place (extremely hot), stoves (scarcely less hot), steam radiators (too hot to be touched) and water radiators (much more comfortable and easier to live with). When walls, floors and ceilings are made the source of heat the temperatures are very moderate, not over 85 F in floors and 110 to 140 F in ceilings.

The term radiant heating, like many of the names invented or chosen by engineers, is largely a misnomer. Radiant heat is that which is propelled through the air from the source by virtue of vibration or heat waves, analagous to sound or light waves. It may be distinguished from heat distribution by conduction or convection. I say "radiant heating" is a misnomer because all the familiar sources emit heat by radiation as well as by convection and conduction. In fact the fireplace emits most of its useful heat by radiation. Possibly the emission of heat from low temperature floors, walls and ceilings occurs more largely by radiation than from radiators (we know that radiators are designed for a

*Presented Feb. 19, 1946 at 42nd ACI Convention.
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rapid circulation of air) and it may be this that gave it the name radiant heating. Some folks call it panel heating.

To radiate heat, the floor, walls or ceiling must themselves be kept warm. This is done by embedding pipes within the surfaces through which warm water is circulated. For efficient operation the material of the radiant surfaces must be a fairly good conductor of heat, and it must have intimate contact with the pipes. Concrete and plaster have been found to be sufficiently good conductors for the purpose, and they are rather easily given close and complete contact with the water pipes embedded in them.

The question then arises how the concrete and plaster in which the pipes are embedded fare when the heat is turned on. Does it crack and fall apart? Would careful engineers be justified in allowing pipes for radiant heating to be placed in reinforced concrete floors on which we depend to support machinery and people? Or would that be inviting disaster? A committee of the Institute is currently wrestling with that question and will no doubt some day announce its judgment under what conditions, if at all, radiant pipes may safely be allowed in structural concrete. I could not, if I would, anticipate the committee's action, but I may present some of the considerations that will be taken into account in reaching its conclusion.

Pipes in concrete are not wholly a new thing. We have long permitted electric conduits in structural slabs, if they were not too large, and other innocuous pipes, presumably devoid of temperature or pressure, were admitted. The provision of our Building Regulations with which we have shackled ourselves in respect to embedding pipes, reads in part as follows:

> Pipes which will contain liquid, gas or vapor at other than room temperature shall not be embedded in concrete necessary for structural stability or fire protection. Drain pipes and pipes whose contents will be under pressure greater than atmospheric pressure by more than one pound per square inch shall not be embedded in structural concrete except in passing through from one side to the other of a floor, wall or beam.

We were a timid lot, were we not, when we wrote that in our standard building regulations. We willingly stress reinforcing steel to 20,000 psi embedded in concrete but a good steel pipe must not have more internal pressure than one pound, presumably in the fear that it might swell and disrupt the concrete even if it did not explode. I fancy we were willing to limit ourselves so drastically because there was no desire to embed pipes in concrete other than electric conduits, small vacuum pipes and low pressure gas pipes. These we admitted, excluding all others without feeling that we were denying ourselves anything of value. With the development of radiant heating, however, it becomes of great value to embed heating pipes in structural concrete and we must scrutinize our prohibitions to make quite sure that we do not needlessly impede useful progress.

Let us inquire what might conceivably happen to concrete containing pipes for radiant heating from which we ought to defend our clients and the clients of our fellow engineers?

1. The pipes might leak.
2. Some one might drill a hole in the floor and puncture a pipe.
3. The concrete might get over warm, dry out and suffer disintegration.
4. The pipes might expand from the heat lengthwise and sidewise and cause the concrete to crack.
5. The concrete might become warmer in some places than in others causing the warm concrete to expand and the cooler concrete to rupture.

Let us discuss these undesirable possibilities one by one, in order.

Of course, if the pipes should leak that would be bad in several ways. Dripping pipes are always an annoyance aside from loss of water from the heating system. Besides if allowed to persist it might in time cause reinforcing to rust and thus endanger the structure. But are pipes embedded in concrete more likely to leak than those which are not? It does not seem so to me. They would inevitably be located nearer the neutral axis than the reinforcing bars and would be subject to only small bending stresses. In any event, if embedded pipes were prone to leak, that condition would prove intolerable, especially considering the expense of repairing such pipes, and our problem would vanish for no one would then wish to place heating pipes in concrete. It is not our task to design pipe installations so as not to leak, nor to appraise the cost of repairs should leaks occur. We need not defend our clients against that risk and should not attempt to do so. Our responsibility is limited to the integrity of the concrete structure.

And it would be equally bad to have a drill driven into a pipe in a reasonable and proper attempt to bolt machinery to the floor or to fasten motors, shafting or other equipment to the ceiling. It may be that in some buildings there will be so much post-construction drilling of floors that use of embedded heating pipes is not advisable. But here again, the problem is not ours. The plant manager should know whether he simply must be free to drill anywhere without taking precautions to avoid pipes. If he must, he had better use some other method of heating. It is his problem to appraise the risk and make the decision. Let me say, however, that I have never known electric conduits to be excluded from a concrete floor for a similar reason.

The next question is whether concrete used in radiant heating may get too warm for its own good. One piece of evidence to the contrary is contained in the Portland Cement Association's *Bulletin ST 32* which recommends that, "Concrete should not be exposed continuously to temperatures much above 500 F." The temperatures anticipated in radiant heating would never exceed 200 F and could be held to a lower

limit without seriously handicapping development of this method of heating. I imagine a good many reinforced concrete structures in the ceilings of boiler rooms, in the foundations of boilers, furnaces and other heat-producing equipment reach this temperature without suffering damage.

Whether concrete would crack too badly from expansion of embedded heating pipes is a question the committee will try to resolve. The expansion lengthwise of a steel pipe when heated 100 F is about the same as the stretch of a reinforcing bar stressed to 20,000 psi. Transverse cracks would be expected to accompany both types of elongation. Need either be taken more seriously than the other? Of course, the stretched rod grows thinner while the heated pipe gets stouter. Perhaps only experience will enable us to resolve our doubts on this question.

The fifth question relates to temperature differentials in the concrete itself. That which is close to the pipes will be warmer than that which is midway between pipes or remote from them. Will the cooler concrete crack seriously enough to endanger the stability of the structure? This is a question the answer to which is not too clear or sure. There is some evidence, however, that the question is not closed against radiant heat. Many sound reinforced concrete structures are subject to differential temperatures more severe than those necessarily inherent in radiant heating. Floors under bakery ovens, and ceilings over them get pretty hot when the ovens are in operation for a long period.

A common example of differential temperatures is found in the floors of reinforced concrete buildings where the edges, sometimes integral with spandrel beams, are exposed to freezing weather and temperatures far below freezing while the same floors are comfortably heated in the interior. Doubtless cracking occurs but not intolerably.

A great many radiant heating installations have already been made with the pipes embedded in concrete. In far the larger number of cases the floor is laid on the ground and does not come under the head of structural concrete. But the fact that no case has been reported of concrete disintegrating or badly cracked from the embedment of heating pipes is evidence favorable to the belief that such embedment may safely be allowed under reasonable limitations in structural concrete.

It is fortunate, perhaps, that our Standard Building Regulations, useful as they are, have not universal application; that there are regions where they may be disregarded. There the heretical and nonconformist engineer with courage and confidence in his own judgment may pioneer and show us the way—the way to take or not to take. For it is often the heretic who makes progress possible for the orthodox. I am not suggesting that the Institute adopt heresy as a principle, only that it refrain from obstructing progress more than necessary.

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Discussion of a paper by John R. Nichols:

Radiant Heating By Reinforced Concrete*

By P. B. GORDON, RAYMOND G. VANDERWEIL, C. A. HAWK, JR., M. B. LAGAARD and BURGESS H. JENNINGS, and AUTHOR

By P. B. GORDON†

I have reviewed with interest the paper of Mr. Nichols discussing the use of reinforced concrete surfaces as the means of creating heating panels for space heating of buildings. Such heating methods have been used in an increasing amount over the past 35 years not only in this country, but in England and on the continent, and the resulting experience has been very satisfactory.

I do not propose to review any of the advantages of panel heating in so far as the heating and comfort aspects are concerned but propose to list a few of the points that may be of interest to a group of engineers concerned with structural concrete design and its relation to panel heating.

As regard the problem of corrosion, I should like to make some additional points beyond Mr. Nichols' paper. The piping material generally used is mild steel though increasing amounts of genuine wrought iron and some copper tubing have been used. In so far as corrosion from the exterior of the pipe is concerned, pipe completely imbedded in concrete is entirely free from corrosion unless there exists an external corrosive agent which would leak through the concrete, affect the pipe and also affect the reinforcing members. It is understood that this limitation is based on the use of structural concrete and not on the use of cinder concrete which is extremely corrosive. The inside of the pipe or copper tubing will also be resistant to corrosion if the heating medium is circulating warm water. A recirculation warm water system with little or no makeup water provides a system inherently free from corrosion. It is strongly recommended that steam or vapor-vacuum

*ACI JOURNAL, April 1946; *Proceedings* V. 42, p. 513
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systems not be used as the heating medium because of this possibility of corrosion.

As concerns tightness beyond leaks that may occur due to corrosion, the other leaks to be considered would be due to faulty pipe material or improper joints. Either of these two faults can be determined in advance of concrete pouring by proper testing procedures. In so far as possible, good practice calls for installing the piping or tubing with as few joints as possible imbedded in the structure. Good practice also requires welding of steel or wrought iron and if copper tubing is used, the few joints that would occur in the concrete can be made by solder fittings.

Tests for the determination of structural strength and tightness are usually provided by means of a 500 psi air test under water on the fabricated pipe coils prior to shipment and a subsequent 250 psi hydrostatic test on the coils and its connected branches after erection but prior to the pouring of encasing concrete. With these precautions, we do not encounter any difficulty.

The stresses set up in the piping system are of a very low magnitude as compared with normal stresses set up in similar materials used for reinforcing in structural concrete. This is due to the very small temperature changes involved and also because the differences in the coefficients of expansion for concrete and steel and concrete and copper are of such a small degree. The resulting tension or compression stresses set up in the piping based on this is but a very small percentage of the allowable stresses permitted in these materials. Also these stresses all are of such a low order that its effect on the cohesive bond between the pipe and concrete envelope is very minor. In line with the above, it should be pointed out that the water temperature variations are usually in the order of 100 F or less, such as from a low of 40 F to a high of 140 F, though some design for concrete structures permits going to a top of 150 F to 160 F.

The usual steel or wrought iron piping used is good for internal pressures of 400 to 800 psi while copper tubing with soldered fittings may be subjected to internal pressures up to 500 psi in the temperature range we are talking about. Since panel heating systems should only use warm water circulation systems and since good practice also calls for an open expansion tank type of distribution, the resulting maximum working pressures encountered are due to the static head created by the height of the top of the system above the lowest coils, plus the dynamic head necessary to create circulation. If we use a 20 story building as a top limit which might be 250 feet high, this would mean that we could expect pressures in the order of 125 to 150 psi as an extreme, which is well below

the allowable safe limits for the structural strength of the pipe or tubing used.

By RAYMOND G. VANDERWEIL*

I want to discuss Mr. Nichols' paper from the point of view of the mechanical engineer. In the first place, I would like to express my complete agreement with Mr. Nichols, in that the progress of the newer sciences should not be obstructed by too much conservative thought, particularly in the field of heating. This field has been essentially stagnant throughout the last three decades. For a great number of structures, such as hospitals, schools, apartments, office buildings and residences, radiant heating approaches nearer to the ideal physiological conditions than any of the other conventional heating systems. The foremost example of comfort conditions could be taken as a clear calm spring day. The air temperature is relatively low and the humidity relatively high. No differential of temperature occurs in the height of a person enjoying such a day, the heat comes directly from overhead, and the surroundings are relatively cool. There is little dust stirred up because of the lack of draft and wind, and the odors are pleasant. All of the foregoing would take place equally in a properly designed radiant heated room. Very little of this would occur in a conventionally heated room supplied with radiators. In a radiant heated room with the panel in the ceiling, the human body receives considerable heat by radiation from above; a parallel to the sun radiation on our clear, calm, spring day. Space requirements for the heating system are reduced to a minimum and there is full architectural freedom in the treatment of the interior of rooms.

Supplementing Mr. Nichols' note concerning the intensity of radiation output, it may be of interest to compare the various systems on the basis of percent of total heat output which is transmitted by radiation.

Type of system	Radiation, per cent, approx.
Air system	10
Radiator system	15
Floor radiant panel	35
Ceiling radiant panel	65

Need it be noted that our earth receives 100 percent of its heat by radiation from the sun?

I would like to give a few words concerning four of the five possible sources of trouble mentioned in Mr. Nichols paper; point 3 being a fully structural problem and not to be discussed.

1. In reference to the possibility of the tubing or pipe leading, it is entirely up to the structural engineer to call for a pressure test rigid

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enough to avoid leaks in the embedded piping system. Even with soldered soft copper tubing, specifications may call for a 300 psi hydrostatic pressure test extending over a period of eight hours or longer. It is almost inconceivable that a system so tested and strictly and visually inspected throughout the test period, will at a later date develop leaks, particularly if the coils are embedded in concrete or plaster shortly after the termination of the pressure test. Many years of experience have proven that occurrence of leaks is no matter of consequence.

2. In reference to the possible damage of the copper tubing by puncture, all radiant heating layouts like other extensive piping systems, should be designed properly and in close cooperation with the architect or the structural engineer.

Where the location of machinery and electrical conduits may be anticipated, the coils should be laid out accordingly. The probability of puncture is thereby reduced. Where heavy machinery is to be shifted in the distant future, it may be advisable to avoid the placing of coils in the floor and it then would be preferable to use ceiling coils.

The installation cost of radiant heating systems in multi-storied structures is relatively low, particularly with the coils embedded near the ceiling surface. Thus extensive use of this type of system may be expected in the future, in office buildings, apartment houses, and particularly in hospitals, where the physiological superiority of this system will be recognized as being essential. In all these installations, puncture of the copper tubing is practically impossible if structural, heating, and electrical engineers coordinate their design properly.

4. The parallel between the pipe and a reinforcing bar stressed to 20,000 psi is highly instructive, and I suppose is based on the "most unfavorable assumption" that the steel pipe's temperature is 100 F in excess of that of the surrounding concrete. However, it should be considered that this most unfavorable case can result only if water of a maximum temperature is suddenly supplied to the panel. This could easily be avoided by proper design of the heating equipment. If the temperature of the panel surfaces is limited to the physiological maximum quoted in the first paragraph of Mr. Nichols' paper, such temperature differences cannot exist if the system is in equilibrium.

Fig. A shows the temperature distribution, in the equilibrium state, of a typical but rather unfavorable concrete slab. Since temperature differences greater than those shown in Fig. A can hardly be expected, the differential between mean tube temperature and mean slab temperature must then be considerably smaller than 100 F. With copper tubing, the differential of expansion coefficients is greater than with

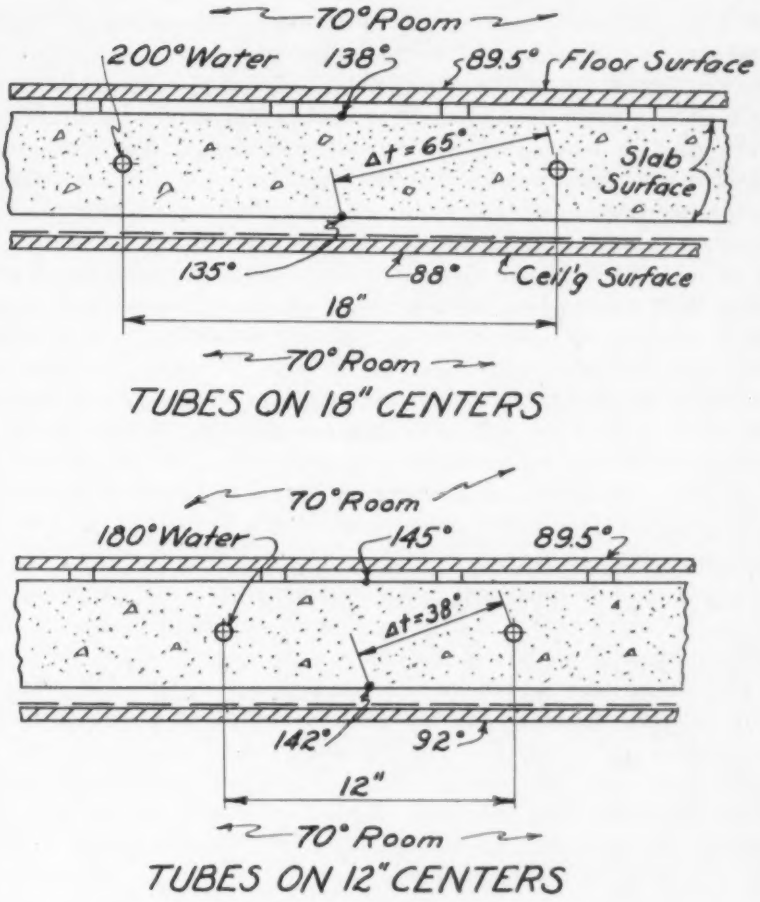

Fig. A

wrought iron or steel but due to the smaller modulus of elasticity of the copper, and smaller tube diameters, the stresses set up in the slab are again of approximately the same magnitude. Several hundred thousand feet of copper tubing, installed in concrete slabs, prove conclusively that the tubing as well as the concrete will readily withstand any stresses set up. An investigation as to the possibility of relatively minor transverse cracks, should be left to the structural engineer, but it may be worthwhile mentioning that no such cracks were found in a great number of systems, where copper coils were embedded in gypsum plaster. Authoritative sources have stated that the cracking patterns in such ceilings are practically identical to that of ordinary ceilings of the same age, without tubing embedded. This is in spite of the fact that no precautions whatever were taken during the heating up period.

5. The question of possible cracks caused by temperature differences in the concrete (see Fig. A) seems rather pertinent. Again practical experience seems to indicate that such cracks, if they occur, are of no consequence. Theoretically, cracks might be expected where the temperature gradient in the slab is steepest; that is, near the tube or pipe. Extensive tests have proven that the bond between tube and concrete is not broken by operation, however, this would not prove the total absence of minor cracks at some point of the slab cross section. It may be worthwhile to run a series of load and deflection tests on panel sections with coils embedded, before and after a relatively long heating period, in order to answer this question thoroughly. By operating several test sections with water of different temperatures, the tests should indicate that critical water temperatures would occur high above the working range of any radiant heating system. In such tests the heated water should be supplied suddenly to the coil of the intermittently heated test section. The condition arising from this would be more serious than actual operating conditions, and we would then be able to have definite information to guide us.

It is worthwhile mentioning that similar tests were undertaken in 1936 at the Engineering College in Prague, (Czechoslovakia). Under the direction of Professor F. Klockner, two concrete slabs, 2 ft. wide and 12 ft. long, were tested for bending resistance. One slab was reinforced by rods, the other one by ⅞-in. OD steel pipe coils heated to temperatures up to 200 F. The purpose of the test was to find the influence of elevated temperatures, cracks and other unknown phenomena upon the strength of the structure. Since the use of tubing as reinforcing members is not permitted according to U. S. practice, the results obtained in these tests are of no consequence.

By C. A. HAWK, JR.*

It is my opinion that the thoughts presented in Mr. Nichols' paper, and the direction in which he is seeking to project the thinking of the Institute, are both sound and progressive. Experience so far seems to indicate that the provisions in the Building Regulations under question are a little too conservative and that proper modification will permit construction of more economical and comfortable reinforced concrete buildings.

The two questions which are left unanswered and which Mr. Nichols feels should be explored further by the Institute are: (1) Will structural concrete crack too badly from expansion of embedded heating pipes? (2) Will the temperature differentials in the concrete itself create serious cracking?

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Obviously, these questions are valid and a need for study along these lines is indicated. It is common knowledge that excessive temperature differentials within structural concrete will damage it severely, but the problem which remains is to determine the safe limits within which temperatures can be raised without endangering the stability of the concrete.

I should, however, like to draw on our very broad experience with radiant heating in all types of structures to point out the fact that the present Regulations have been violated so many times without any damaging effects whatever, that the question confronting the Institute researchers is, "Where are the proper engineering limits?" not, "Can it be done?"

In any case, I am certain that future studies along this line will be both revealing and useful.

By M. B. LAGAARD* and BURGESS H. JENNINGS†

The discussion Mr. Nichols' paper is particularly timely, as many such installations are in progress or being planned at the present time.

Here, as elsewhere, the final test of the method is the test of service; and, in view of the meager amount of performance results, it seems desirable to consider the possibilities of this system from past experiences with somewhat parallel cases.

There are many phases of the question to be considered, such as initial cost, efficiency of operation, maintenance, effect on the structure, advantages of concealment, uniformity of temperatures, and possibilities of partial use as a cooling system.

Metal pipes have been embedded in concrete almost from the time this type of construction was introduced. The effects are well known to all engineers who have been concerned with them. A cast iron light pole or a man-hole casting embedded in a concrete sidewalk is likely to cause radial cracking of the walk unless a cushion of expansion material separates the concrete from the metal unit. Corrosion of hand rail pipes, reinforcing steel, etc., too near the surface is a well recognized cause of spalling and is considered in design.

Plaster coats around large pipes or structural members may crack badly. On the other hand, many conduits, water pipes, etc., have been embedded in concrete in great numbers with no ill effects at all. However, in these cases significant temperature differences were not involved.

Two cases may be cited where considerable differences in temperature between the pipes and the concrete have occurred. One is in Boulder

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Dam where an elaborate pipe system was installed to cool the concrete and the other is where pipes have been embedded in concrete refrigerator walls and display counters of meat markets for cooling purposes. In the former case, no trouble was expected and none has developed. In the latter case, however, serious spalling and disintegration of the concrete has occurred. The adverse results in the second case may be accounted for chiefly by the fact that numerous and rapid changes in temperatures occurred from above to below freezing.

Radiant heating presents a somewhat different problem. High temperature differentials between the pipes and the concrete should be anticipated and thorough drying out of the concrete would be expected.

Considering the question of temperature differentials, it seems quite possible that, inadvertently, temperatures of the pipe, with hot water heating, might easily rise to 180 F, before the surrounding concrete had had much of an opportunity to follow. If such was the case, a temperature differential of 50 to 100 F might occur. This is equivalent to a change in length of $\frac{3}{8}$ to $\frac{3}{4}$ of an in. per 100 ft. In addition to this, a shrinkage of $\frac{3}{4}$ in. per 100 ft. or more could develop from complete drying of the concrete which would augment the bond strain developed by the temperature difference. Failure of the bond between the concrete and the pipe might thus easily occur. However, the bond between the concrete and any nearby reinforcing bars should not be affected because the heat from the concrete would warm up the bar at about the same rate as its own temperature increased.

In case the pipes would also be used for cooling, the condition would be reversed and shrinkage stresses would be relieved.

The question of bond between the pipes and the concrete is brought out because already a number of designers have suggested using the heating pipes as reinforcement on unimportant work where welded joints are to be used.

Where no such use was intended, a bond failure around the pipes would not be objectionable. The reduction of concrete area due to the presence of the pipes in a beam or slab, would, of course, have to be considered in the design.

The question of cracking due to drying out of the concrete deserves special mention. Surface crazing due to a difference in shrinkage between the surface and the interior of the mass is of common occurrence, although not always noticeable to the observer. In compression, the cracks merely close up and the stress is transmitted between the contact surfaces. In direct tension or extreme fibre tension due to bending, the surface cracking weakens the concrete. That is why a beam that has dried out is often weaker than one that has been kept moist, even though drying increases the compressive strength.

High temperatures tend to increase crazings. Very high temperatures will cause wide cracks to develop and eventual disintegration will result. Several cases may be cited where this has occurred. High temperatures therefore should be avoided.

Leakage of the pipes, corrosion, puncturing, etc., are of secondary structural importance and can be taken into account by the proper selection of materials and construction methods.

From a viewpoint of heating, surface temperatures of concrete floors should not be allowed to exceed 80 F and preferably should be kept at lower values. Under these conditions water temperatures in the heating pipes embedded in the concrete should never have to exceed 120 F. However, installations in which water temperatures reaching 135 F and operating continuously at that temperature have been found satisfactory with no deleterious effects on the concrete. Even in this temperature range sudden temperature changes should be avoided. Although use of heating pipes in floor coils has been popular, from a human comfort and convenience viewpoint it seems preferable to place the heating surfaces in the side walls or ceiling of a room when radiant heating is to be used.

In summarizing, the writers are of the opinion that radiant heating through a system of pipes embedded in the concrete can be safely utilized in reinforced concrete work, if the temperatures are not too high, if the temperature changes are not too rapid, and if full consideration is given to the presence of the pipes in the structural design.

AUTHOR'S CLOSURE

I am grateful for the discussions of my paper, which both testify to the existing interest in the subject and extend the facts and opinions, often expert, that are presented. I can add very little more at this stage.

It ought to be pointed out, I think, that the cracks and other evidence of mild or total concrete failure reported by Messrs. Lagaard and Jennings have rather obvious causes which are wholly absent in the ordinary case of radiant heating. The castings embedded in a concrete sidewalk which cause radial cracks are exposed to the weather. No one hesitates to use steel or cast iron frames embedded in floors or walls inside a building for fear of such cracks. If they occur there are of no importance. To the case of pipes in concrete refrigerator walls which went to pieces, might be added the floors of skating rinks. In both cases the presence of moisture and frost would adequately account for disintegration. In the case of skating rink floors the wonder has been that they lasted as long as they did.

If a radiant heating system were to be used for cooling in summer, it would doubtless be desirable to keep the pipe temperature above the dew point to avoid condensation. However, radiant heating has enough to recommend it without using the system for cooling; else there would be no occasion for these present considerations.

I feel sure Mr. Hawk will not object to my quoting from a letter which he wrote me in July, and thus adding a bit to his discussion. In reference to a proposal for heat control made in committee, he wrote "While the above points apply only to the heating problem it is apparent that your primary consideration was structural stability and that the conclusion was based on the assumption that rapid surges of hot or cold water would damage the concrete. There is certainly no gainsaying the necessity of investigation of this point, but I have slowly come to the conclusion that for hot water work it is practically impossible to create a set of conditions that will damage concrete. Steam is another matter, but only because a given pipe size will carry considerably more heat under practical pressures than will water under similar practical limitations. [He might have added the suddenness with which steam brings a pipe from 65 F to 212 F. J.R.N.] However, in all the hot water installations with which we have been connected—and they now number many thousands—we have found absolutely no evidence of any kind of slabs damaged by temperature changes or control characteristics. This experience includes ground slabs and structural slabs, thin and thick, with 'on-off' and 'anticipating' control, in severe climates and mild, and the only conclusion which can be drawn is that the thermal capacity of the concrete slabs is so great that the heating pipes simply cannot carry heat to or from the concrete mass sufficiently fast to create distorting stresses which will damage the concrete." And, besides thermal capacity he might give the concrete credit for the considerable toughness and resiliency which it has in surprisingly large measure.

The testimony of Messrs. Hawk, Vanderweil and Gordon to the long record of radiant heating installations with apparently universal satisfaction at least so far as integrity of the embedding concrete is concerned, is to me most impressive.

I am surprised that no one has suggested that heating pipes in concrete floors would be useful in warming the concrete for setting in cold weather. Besides the obvious advantage of preventing freezing of the concrete, this procedure would have another desired result. It would set up temperature differentials in the plastic, no-stress concrete the effect of which would persist when it became hard. Subsequent cooling would introduce compression in the concrete, tension in the pipes, which would, in turn offset the later effects of turning on the heat, apprehension of which gives rise to all this discussion. Question is whether speci-

fication for this preheating can be drawn with sufficient precision to justify in itself, if so much justification seems necessary, relaxation of the present prohibitions of the Building Regulations. But that is a question for the Committee.

[To facilitate selective distribution, **separate prints** of this title (42-20) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Aug. 15, 1946.]

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The Expansion Test as a Measure of Alkali-Aggregate Reaction*

By R. F. BLANKS† and H. S. MEISSNER‡
Members American Concrete Institute

SYNOPSIS

Following the discovery that some aggregates are acted upon by cement of inordinate alkali content, it has become popular to make expansion measurements on specimens, incorporating the offending materials, subjected to curing in closed, moisture-laden containers. From the inherent limitations of such a test and the variety of ways in which it has been conducted, there has accumulated much conflicting data, the differences of which are discussed and recommendations given for greater tolerance in the interpretation of results. A procedure is proposed for appraising the reactive potentiality of prospective aggregates.

The expansive deterioration of concrete through reaction between aggregate and high-alkali cement has been the subject of numerous published articles during the past four years. Prominent in most of these papers are discussions of investigations made with mortar bar expansion specimens, incorporating aggregate and cement of high- to low-alkali content. These specimens are equipped with reference gage points with which to measure length changes and are generally cured within sealed containers in the presence of moisture. It has become apparent that minor details in the procedure and technique of fabricating and curing such specimens have produced conflicting data and anomalous results, leading to widespread misinterpretation and general confusion regarding the value and purpose of this test. Such misapprehension is reflected in a series of discussions appearing under the title of "Notes on the Effect of Alkalies in Portland Cement on the Durability of Concrete" in the appendix to the 1943 report of Committee C-1 of the American Society

*Received by the Institute, Dec. 27, 1944.
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for Testing Materials.* It is the purpose of this paper to clarify some of the factors which affect the test and which require consideration in correctly interpreting the results, and to propose a uniform procedure for testing the potential activity of untried cement-aggregates combinations.

The mortar bar expansion test has become the favorite procedure for studying the activity between cement and aggregate. Through its popularity and widespread use, with the inevitable introduction of variations, several organizations, including the Bureau of Reclamation, have experienced apparent anomalies, inconsistencies and limitations in the mortar bar test. The extensive use of this test, as the only available procedure for predetermining troublesome combinations of cements and aggregates, has provided a large volume of data and information which show conclusively that the test as presently (and variously) performed does not yield results which can be used comparatively at ages beyond a few months. Expansion at later ages may be largely or entirely induced by factors wholly independent of the reactive tendencies of the particular combination of materials involved. Some factors, involved in test procedure, which influence the results obtained with the mortar bar expansion test are:

1. Development of cracking
2. Temperature
3. Admixtures
4. Amount and size of reactive aggregate
5. Cement content
6. Fineness of cement
7. Moistures and storage conditions

1. DEVELOPMENT OF CRACKING

Some investigators have concluded that the comparative value of the expansion test is destroyed when the bars develop severe cracking. The supporting theory is based on the deduction that cracking materially alters the forces causing expansion. When a bar expands to a degree which is accompanied by severe cracking, it has served its primary purpose and should no longer be compared quantitatively with other specimens which have not deteriorated to a comparable degree. The Portland Cement Association has put the above theory into practice by discontinuing readings after an expansion of 0.15 percent has developed.

Various investigators have observed a recovery in sonic modulus of bars following a decline accompanying expansive deterioration (such as illustrated in the upper portions of Fig. 10 and 12). This recovery has been explained as the result of an autogenous healing action or deposition of

*Proceedings, Am. Soc. for Testing Materials, Vol. 43, 1943, p. 199 (An excellent bibliography on the subject of alkalies in cement is to be found on p. 219 of this reference).

Fig. 1—Exudations of sodium silicate gel forming bead-like deposits along cracks in a mortar bar containing reactive aggregate and high-alkali cement.

carbonates, gels, or other cementitous substances in the cracks. In this connection the photograph, shown in Fig. 1, of gel exudation along cracks in a test bar is of interest, although the phenomenon may be variously interpreted. The extent to which healing or recovery may occur seems to be associated with the severity of cracking, bars with fine cracks recovering more rapidly and more completely than those with large, open cracks. Fig. 2 and 3 illustrate a severely cracked 3- by 3- by 15-in. concrete bar, which had expanded 1.24 percent after 15 months in moist, sealed storage. A satisfactory explanation of how or why such large open cracks develop at the surface yet extend into the concrete only a short distance has not been developed. This condition is typical of concrete in service which has been affected by alkali-aggregate reaction.

Severe deterioration of bars is usually accompanied by varying degrees of warping and in some cases by local displacement of inserts as illustrated in Fig. 4. Both of these factors vitiate close comparison of expansion test results.

2. TEMPERATURE

It generally has been found that increased storage temperature, up to about 130 F, accelerates the expansion of bars made with reactive combinations of cement and aggregate, although reverse effects and incon-

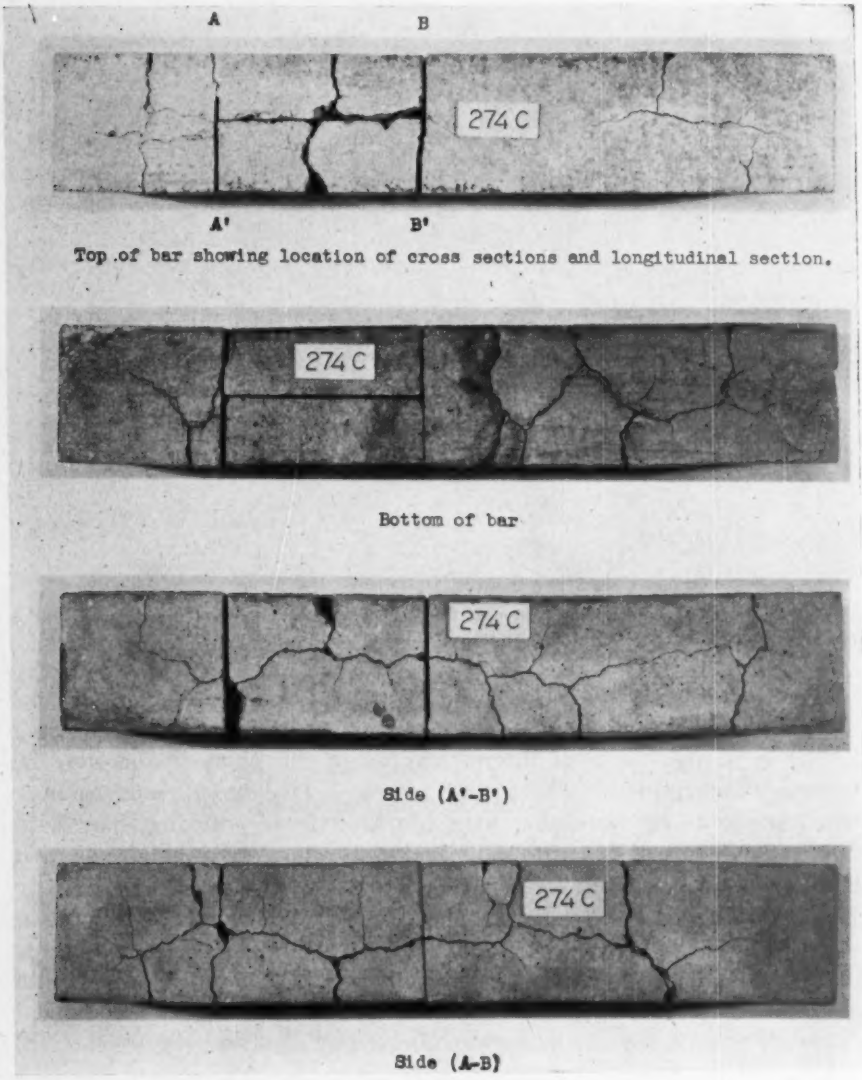


Fig. 2—Appearance of a 3- by 3-by 15-inch concrete bar, containing reactive aggregate and high alkali cement, after 15 months storage in humid cabinet. Cuts were made with diamond saw and photos of section appear in Fig. 3.

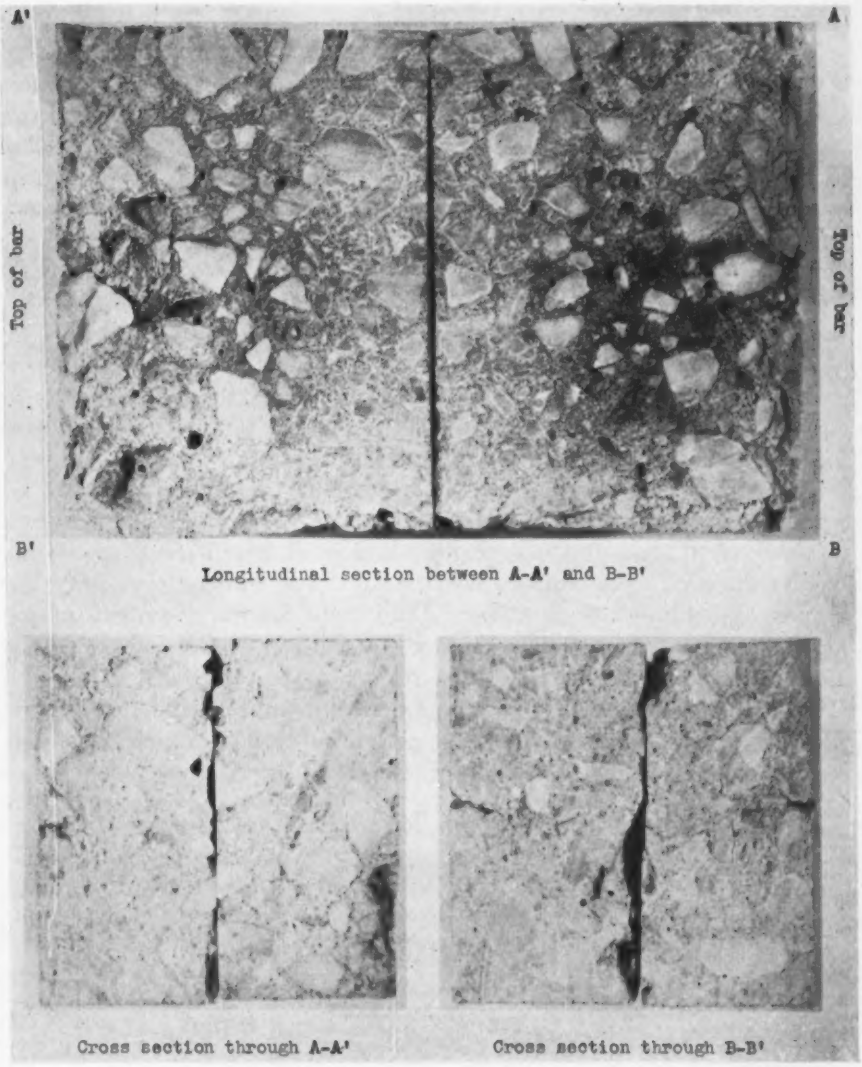


Fig 3—Sections cut through bar shown in Fig. 2, revealing shallow depths of cracks

sistencies have been reported. Tests conducted by the Bureau of Reclamation show fairly consistent increases in expansions for storage temperatures of 70 F., 100 F., and 130 F. Results obtained at temperatures higher than 130 F. appear to be unreliable. Stanton has concluded that temperatures of 100 F. to 110 F. provide most satisfactory acceleration. Tremper finds lower expansions at 100 F. than at 70 F. for certain conditions and combinations of materials. Carlson concluded that temperatures of about 110 F., produce maximum expansions. Explanations for all the inconsistent effects of temperature are not apparent, but the results do show that this method for accelerating the reaction between high-alkali cement and aggregate must be used with caution.

Fluctuations in temperature cause variations in relative humidity within the storage space which will in turn affect the amount of moisture deposited on the test specimens by condensation. The significance of this factor will be discussed under a later heading.

As a matter of convenience, it is the usual practice when tests are conducted at elevated temperatures to reduce the temperature to normal room level to make test observations and readings. If readings are not made at normal room temperature, the control of drying and other factors impose difficulties. Such alterations of high and low temperature produce an effect, called "ratchet action" by Carlson, which induces permanent expansion independently of reaction between cement and aggregate. Thermal expansion with rising temperature apparently effects slight changes in the orientation of the granular particles which remain in the changed positions when the temperature is later reduced. Repeated

Fig. 4—Warping and local displacement of inserts are factors which may vitiate comparative results from expansion tests.

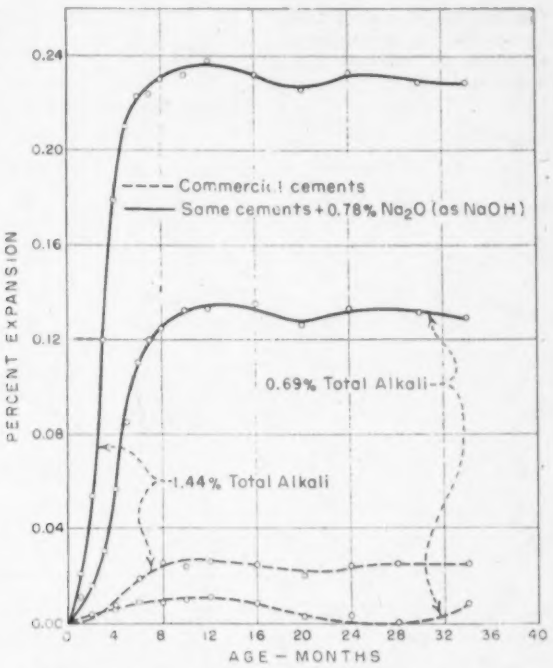


Fig. 5—Addition of sodium hydroxide to a non-reactive combination increases expansion.

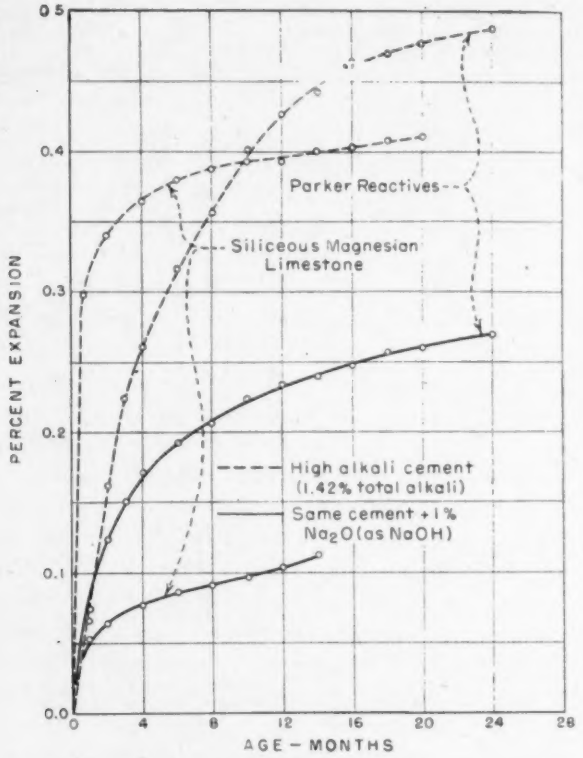
1:3 Mortar. 1-by 1-by 10-inch bars. Stored sealed moist at 100 F. Kutras sand with cements as shown.

alternations result in permanent elongation and enlargement of unrestrained specimens.

3. ADMIXTURES

A number of admixtures have been used in combination with reactive aggregate and cements of varying alkali contents. Sodium hydroxide with high alkali cement and a highly reactive aggregate frequently has been found to reduce measured expansion, possibly because more of the adverse reactions take place while the concrete is still plastic. The addition of sodium hydroxide to a nonexpanding combination of low-alkali cement and reactive aggregate will usually induce expansion. The inconsistent results obtained by various investigators lead to the conclusion that addition of sodium hydroxide does not provide a very satisfactory method for accelerating the expansion bar test. Some results obtained in the Bureau of Reclamation laboratories using sodium hydroxide as an admixture are shown in Fig. 5, 6 and 7, illustrating the foregoing statements. When sodium hydroxide is used with a highly reactive opaline shale, Fig. 7, it is observed to increase or decrease expansion, depending on the size and quantity of reactive aggregate used in the mix.

Fig. 6—Addition of sodium hydroxide to a reactive combination reduces the expansion



1:2 Mortar. 2-by 2-by 10-inch bars. Stored sealed moist at 100 F. Expansion shown is the average on 27 bars, three each for nine mixes in which crushed reactive aggregates were used in sizes No. 8, No. 30 and No. 100, each size being used in 4.75, 9.5 and 19 percent to replace that part of crushed non-reactive quartz.

Various pozzolanic materials, when used as replacements for part of the cement, have been shown to have a beneficial effect on the expansive deterioration of reactive cement-aggregate combinations. For such materials, the reduction in expansion normally varies as the percentage of replacement. In this connection, it is of interest to note that a sufficient quantity of finely ground reactive aggregate will either reduce or eliminate expansion, as illustrated in Fig. 8, possibly because the adverse reactions take place sooner, while the concrete is still plastic and while it can accommodate the expansive forces.

Calcium chloride increases the expansion of specimens made with reactive materials, whereas aluminum powder decreases such deterioration. (See Fig. 9 and 10.) The use of aluminum powder as a corrective for alkali-aggregate deterioration holds little promise, however, because the quantity required to effect material improvement also causes marked decrease in strength and density. The data presented in Fig. 11 indicate

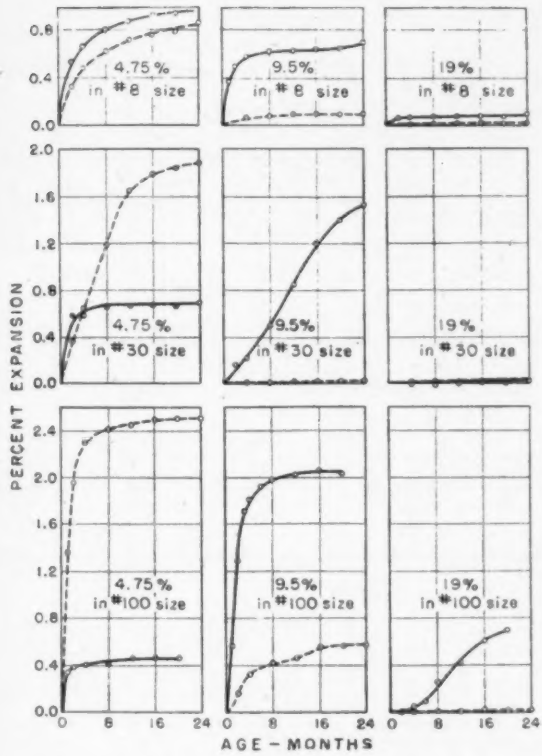

4.75 % in #8 size
9.5% in #8 size
19% in #8 size
4.75 % in #30 size
9.5% in #30 size
19% in #30 size
4.75 % in #100 size
9.5% in #100 size
19% in #100 size
PERCENT EXPANSION
AGE - MONTHS

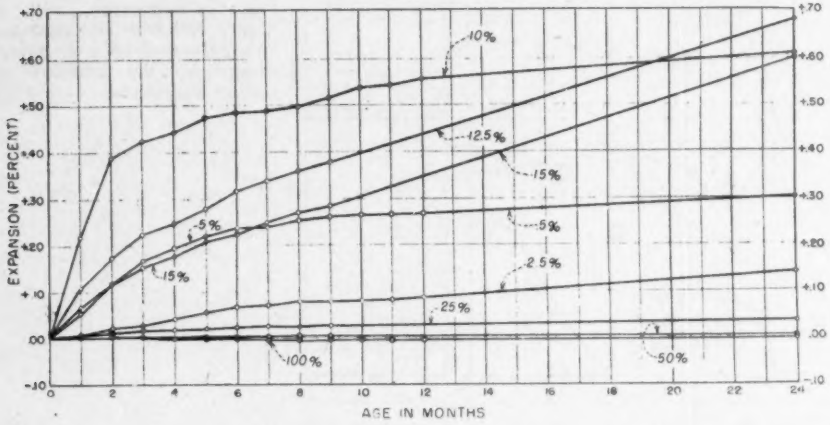
Fig. 7—Expansion, with a highly reactive combination, is accelerated as well as retarded by the addition of sodium hydroxide.

1:2 Mortar. 2-by 2-by 10-inch bars. Stored sealed moist at 100 F. Crushed Opaline Shale used in the sizes and per cent of the total aggregate as shown, to replace an equal part of nonreactive quartz.
—High alkali cement (1.42%)
—Same cement + 1% Na_2O (as NaOH)

that the beneficial effects of aluminum powder are due to its aerating properties, because the benefits are lost when the powder is ground with the cement in the presence of small quantities of water. The aggravating effect of calcium chloride is of interest because of the general use of this material in concrete as a strength accelerator.

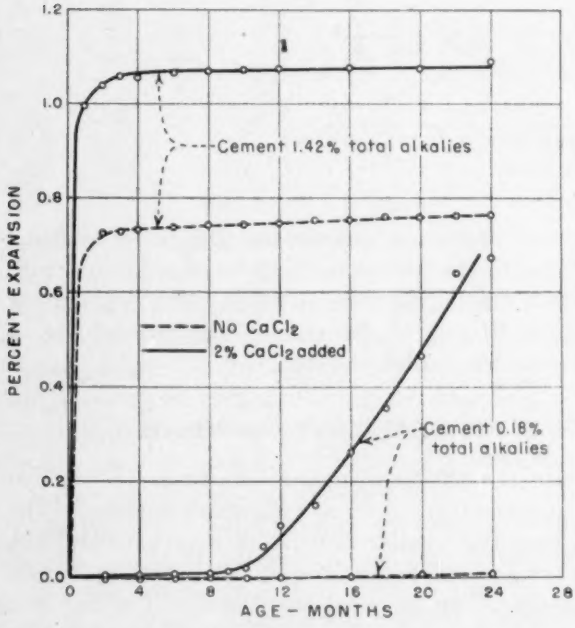
4. AMOUNT AND SIZE OF REACTIVE AGGREGATE

Fig. 12 and 13 illustrate the effects of amount and size of reactive aggregate on expansive deterioration with a high-alkali cement. The results are consistent in that the smaller the size of aggregate and the larger the quantity, the greater the expansions for all sizes larger than No. 100, and for all percentages up to about 10 percent. The fine size apparently acts as a pozzolan, as mentioned previously. This effect is more definitely indicated by the results in Fig. 8 wherein it is shown that,



Each curve represents an average of the results from three 3-by 3-by 15-inch concrete bars stored in humid cabinets in which pure quartz aggregate was replaced in all eight sieve sizes, from pan(—No. 100) to ⅜"-¾", by siliceous limestone in the percentage shown. Mix 1:2.72:2.72, ¾" max. concrete, W/C=0.60 (proportions by weight), cement content 1.50 bbls. per cu. yd. Cement contains 1.38% alkali ($Na_2O + .658\ K_2O$).

Fig. 8—Expansion is greatest when the reactive aggregate is present in a "pessimum" amount



1:2 Mortar. 2-by 2-by 10-inch bars. Stored sealed moist at 100 F. Reactive material is 19 percent of No. 100 siliceous magnesian limestone replacing pure quartz.

Fig. 9—Addition of calcium chloride increases expansion of reactive mixtures

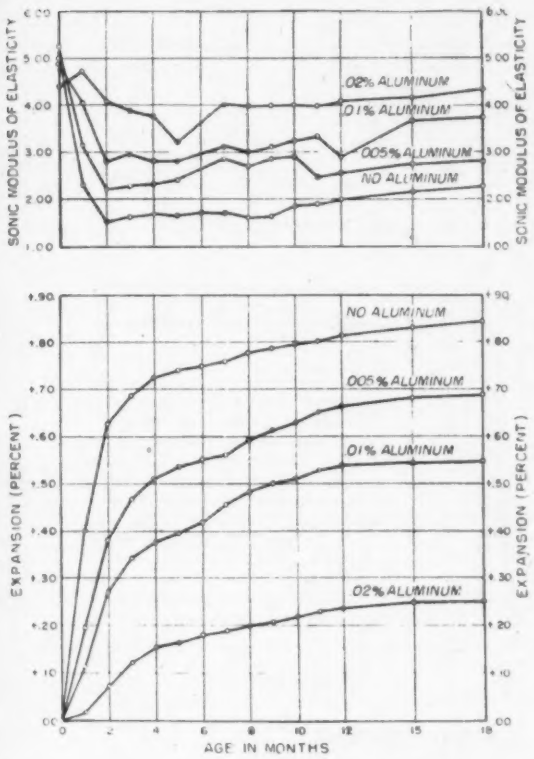


Fig. 10—Effect of addition of aluminum powder on expansion and sonic modulus of 3- by 3- by 15-inch concrete bars stored in humid cabinets.

Each curve represents an average of the results of three bars in which the No. 50 and No. 30 sieve sizes of the pure quartz (9.5% of the total aggregate) was replaced by an equal weight of reactive siliceous magnesian limestone. Unpolished aluminum known as "Alcoa Albron" No. 761 was dry-mixed with the cement. Aluminum powder additions as shown on the curves, are expressed as the percent of cement by wt. Mix 1:2.72:2.72, ¾" max. concrete, W/C = 0.60 (proportions by wt.) Cement content 1.5 bbls. per cu. yd. Cement No. 2742-Na_2O = 1.30% KO_2 = 0.12%, Na_2O + .658 K_2O = 1.38%.

added in sufficient quantity, the reactive aggregate provides its own curative.

The effect of size and amount of reactive aggregate appears to be different for different cements, as indicated in Fig. 14. In studying the results shown in this diagram it should be borne in mind that there are wide differences in other properties of the various types of cement represented, and that comparisons solely on the basis of alkali contents should be made with caution. However, it is significant that no expansion has developed with low-alkali cements.

It is very difficult to obtain a true comparison of the effect of size of aggregate because of contamination by flour material. Fig. 15 illustrates such contamination in a very carefully screened aggregate fraction. Thorough washing is required to eliminate this factor.

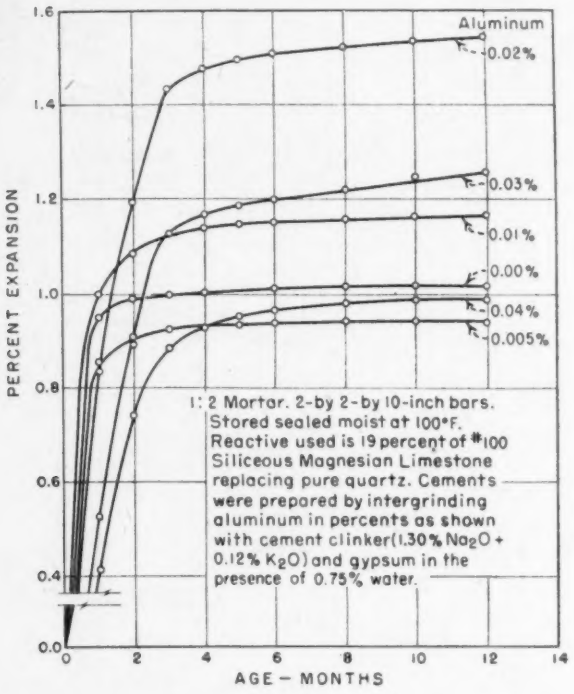

Aluminum
0.02%
0.03%
0.01%
0.00%
0.04%
0.005%
PERCENT EXPANSION
AGE – MONTHS
1:2 Mortar. 2-by 2-by 10-inch bars. Stored sealed moist at 100°F. Reactive used is 19 percent of #100 Siliceous Magnesian Limestone replacing pure quartz. Cements were prepared by intergrinding aluminum in percents as shown with cement clinker(1.30% Na_2O + 0.12% K_2O) and gypsum in the presence of 0.75% water.

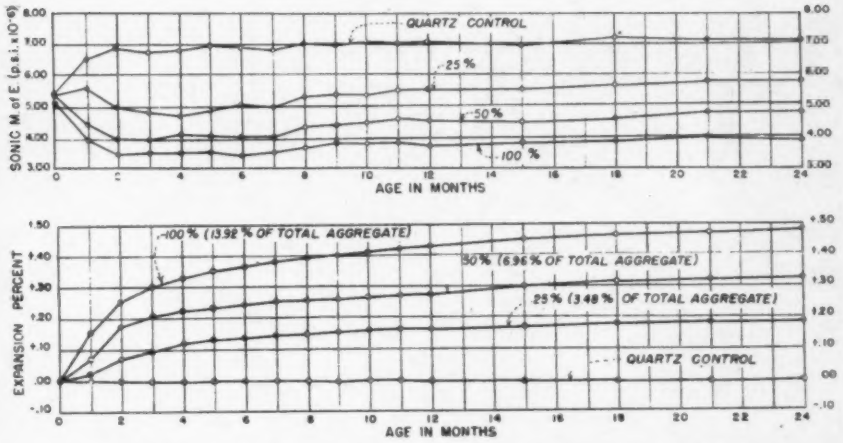
Fig. 11—Aluminum and water interground with high alkali cement may increase expansion with a reactive aggregate.

5. CEMENT CONTENT

Increasing the richness of the mix, within practical limits, also accelerates expansion as reflected by the curves plotted in Fig. 16. This effect appears to be entirely logical because increasing the cement content also increases the quantity of alklalies that can react with the aggregate.

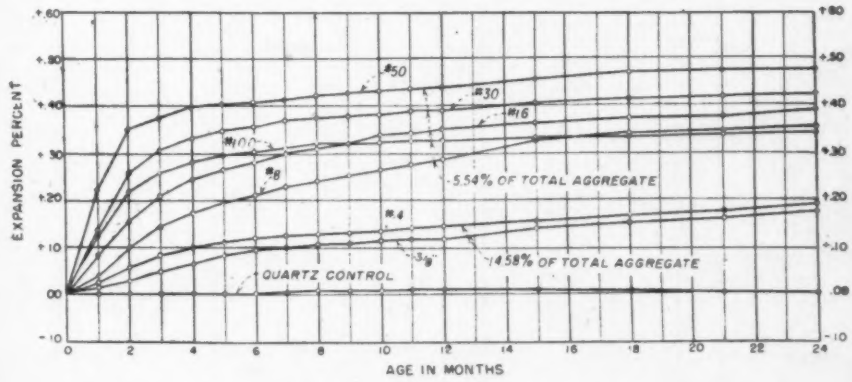
6. CEMENT FINENESS

It is logical that, if the size of reactive aggregate can influence the amount of expansion occurring with a high-alkali cement, the fineness of the cement should likewise be found to have an effect. The alkalies will be released from fine cement more rapidly than from a coarse ground cement and hence will be more readily available for combination with a reactive aggregate. That this is true is evident from Fig. 17, wherein is shown the expansion secured with three reactive aggregates combined with a high-alkali cement ground to two different finenesses. In each case the expansion resulting from the use of coarse cement is negligible or materially less than that produced by the same cement ground to higher fineness. These data reflect the effect of cement fineness on early expansion only, and it may be presumed that as the coarse cement continues to hydrate, greater expansion will eventually develop. Neverthe-



Each curve represents an average of the results from twenty-one 3-by 3-by 15-inch concrete bars stored in humid cabinets, 3 for each of 7 mixes, in which pure quartz aggregate was replaced in each of 7 sieve sizes, from No. 100 to ⅜", by 25%, 50%, and 100% reactive siliceous magnesian limestone. Mix 1:2.72:2.72, ¾" max. concrete, W/C = 0.60, (proportions by weight), cement content 1.50 bbls. per cu. yd. Cement contains 1.38% alkali Na_2O +.658 K_2O). Entire series duplicated with another cement containing 0.13% Na_2O +.658 K_2O, no expansion with this cement up to 24 months.

Fig. 12—The expansion and reduction of elastic modulus of concrete bars becomes greater as more reactive aggregate is used in the mix.



Each curve represents an average of the results from nine bars, three for each of three mixes in which a given size fraction of pure quartz aggregate was replaced by 25%, 50% and 100% reactive siliceous magnesian limestone. Mix 1:2.72:2.72, ¾" max. concrete, W/C = 0.60 (proportions by weight), cement content 1.50 bbls. per cu. yd. Cement contains 1.38% alkali (Na_2O +.658 K_2O). Entire series duplicated with another cement containing 0.13% Na_2O +.658 K_2O, no expansion with this cement up to 24 months.

Fig. 13—Expansion of 3- by 3- by 15-inch concrete bars stored in humid cabinets is greatest with reactive aggregate sized to pass No. 30 screen and retained on No. 50 screen.

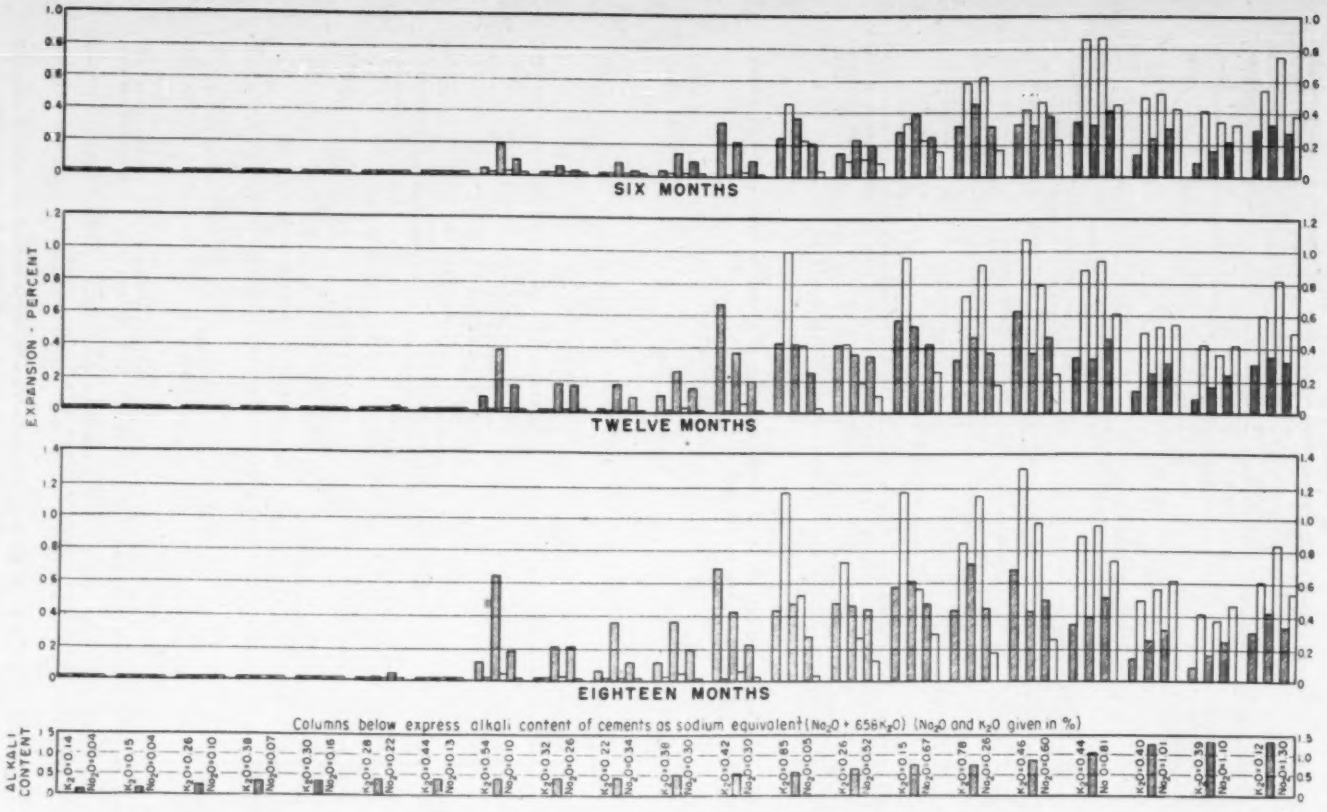

SIX MONTHS
EXPANSION - PERCENT
TWELVE MONTHS
EIGHTEEN MONTHS
Columns below express alkali content of cements as sodium equivalent ($Na_2O + 658K_2O$) (Na_2O and K_2O given in %)
ALKALI CONTENT

Fig. 14—Expansion of concrete is affected by alkali content of cement and by size and amount of reactive aggregate

Each column represents the average results from 3—3 x 3 x 15 inch. specimens in which pure quartz aggregate was replaced by siliceous magnesian limestone. Mix proportions — 1:2.72; 2.72, W/C = 0.60. Cement content = 1.50 bbls. per cu. yd., ¾" maximum size aggregate. Solid columns are 50 percent replacement and open columns 100 percent replacement of quartz with limestone in one sieve size. Left hand two columns in each group are replacement in size No. 100—No. 50, two center columns, replacement in size No. 50—No. 30, and righthand two columns, replacement in size No. 16—No. 8.

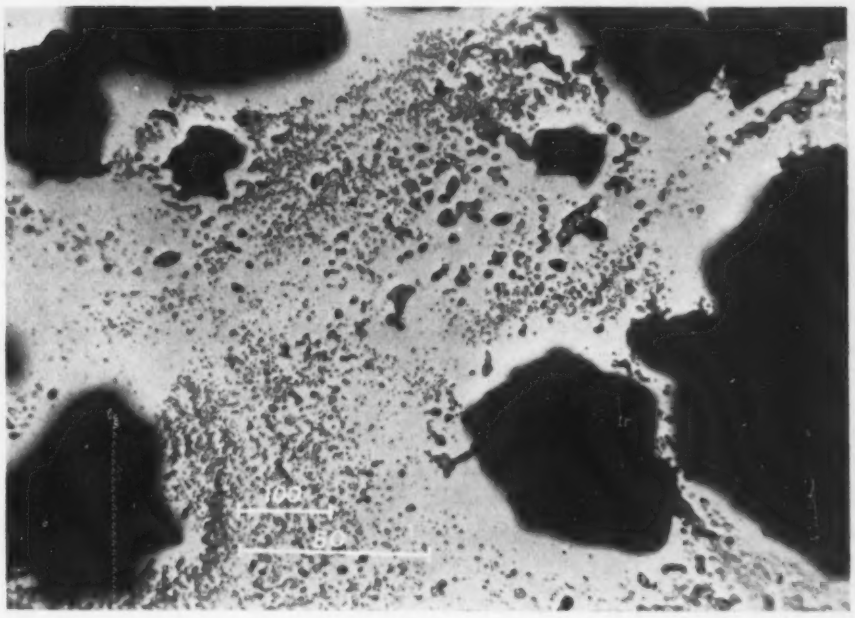
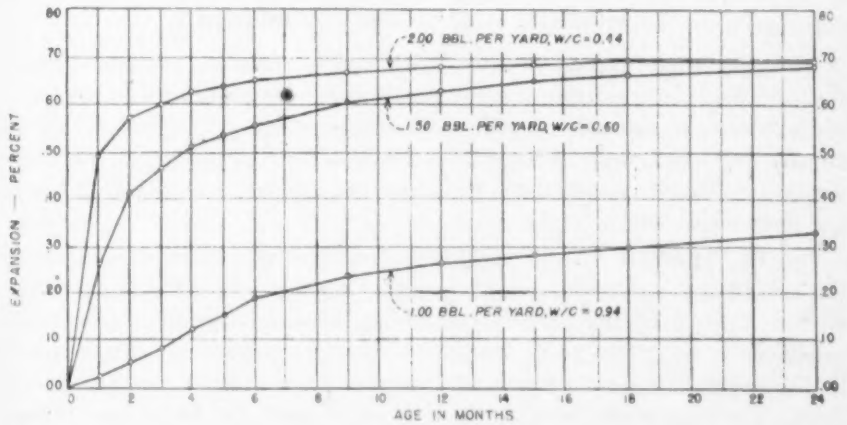


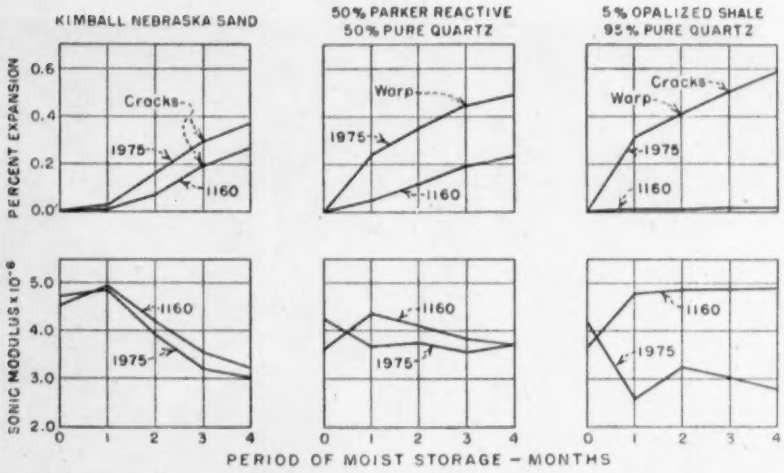
Fig. 15—Unless aggregate screening is accomplished by very thorough washing, a given size fraciion will contain varying amounts of undersize and "flour" material. (Buck phyllite, passing 50-mesh and retained on 100-mesh. Sizes of screen openings are shown)



Each curve represents the average of the results from nine bars for each of the cement contents shown, three each of three mixes in which pure quartz aggregate was replaced in the No. 100—No. 50, No. 50—No. 30, and No. 16—No 8 sieve sizes by 100% (9.5% of the total aggregate) of siliceous magnesian limestone. Cement contains 1.38% alkali ($Na_2O + .658\ K_2O$). Entire series duplicated with another cement containing 0.13% $Na_2O + .658\ K_2O$; no expansion with this cement up to 24 months.

Fig. 16—Expansion of 3- by 3- by 15-inch concrete bars, stored in humid cabinets, is greater for mixes with higher cement contents.

replacement in size No. 16—No. 8, No. 50—No. 30, and right-hand two columns, replacement in size No. 16—No. 8.



Cement ground to fineness as shown using clinker (1.30% Na_2O + 0.12% K_2O). 1:2 mortar bars. 2-by 2-by 10-inch. Sealed with moisture at 100 F. Initial readings at 7 days age.

Fig. 17—Fine grinding of a high alkali cement accelerates its expansion with a reactive aggregate.

less it is demonstrated that cement fineness influences the activity between high-alkali cement and reactive aggregate and this fact may account for the lack of concord found by investigators when attempting to correlate expansion with alkali content of various cements used.

7. MOISTURE AND STORAGE CONDITIONS

The details pertaining to the condition of bars during storage that influence the expansion-test results, for a given combination of materials, are probably more numerous than all others combined. Most details of storage conditions affect either directly or indirectly the moist condition of the bars, which, as Stanton originally pointed out, is a critical factor in the expansion test.

Because moisture effects are controlled to a considerable degree by the size of test specimen, consideration of the various dimensions of the mortar bars in current use is pertinent. Stanton's original tests were made on 1- by 1- by 10-in. bars and most investigators have followed his lead in this respect. Some laboratories use a 1- by 1- by 5-in. specimen. Tests have also been conducted on 2- by 2- by 10-in. bars. Test bars in which concretes have been used instead of mortar have varied in size from 3- by 3- by 15-in. to 6- by 6- by 36-in.

The popular method of storing mortar bars is in sealed containers, accommodating a variable number of specimens. The air in the containers is kept moist by a small amount of water in the bottom. Some

laboratories intentionally place the lower ends of test bars in the water while others meticulously specify that the specimens shall not be in direct contact with the water. Various methods have been employed to prevent immersion of the bars, such as supporting them on the stainless steel inserts which extend beyond the end of the mortar bar proper, standing them on racks of various materials which are in turn supported in the water by any convenient method, and placing extensions of rubber or other materials on the end inserts.

It seems to be a general practice to store the containers holding test bars in rooms of fairly constant temperature although apparently no particular importance has been assigned to this factor. As previously pointed out, temperature variations have a marked effect on the formation of condensed water.

In the Bureau of Reclamation laboratories, mortar bars are placed in 5-gal. paint buckets, fitted with sealed full-section lids and are stacked in rooms maintained at either 70 F. or 110 F., ±1 F. Bars are supported above the small supply of water in the bottom of the container by rubber extensions on the end inserts used in measuring length changes or on racks welded to the sides of the container. Free water has been used in the bottom of the container but more recently this procedure has given way to use of saturated sand. The individual bars are kept from touching each other by separating grids near the tops of the specimens. Concrete bars are stored in carefully constructed metal cabinets fitted with sealed doors, "wick" curtains, and racks on which the bars are stacked 3-deep, alternating with shallow pans of water. The cabinets are set in rooms having closely controlled temperature.

Even under the most closely controlled conditions obtainable, as to moisture and temperature, wide differences in the actual conditions of the test bars have been observed. In general, the mortar bar will be covered with drops of free moisture from condensation which continually runs down the length of the bars, accumulating leached substances as it drains back to the bottom of the containers. At times the bars in a given container will appear relatively dry. In some of the concrete specimen cabinets the bars will be quite dry, in others they will be covered with free moisture. Radiant heat from lights, the presence of doors, outside or inside walls, floor insulation conditions, air currents, and many other seemingly minor factors appear to influence the moist condition of the specimens.

Analyses of water in the bottom of the mortar-bar containers illustrate how the leaching action of condensed moisture draining down the specimens gradually builds up a contaminated condition. See Table 1.

Analyses of water in the reservoir pans of the concrete bar cabinets show

TABLE 1—ANALYSIS OF WATER IN BOTTOM OF MORTAR-BAR CONTAINERS

Container No.	Types of Cements Represented	Analysis of water (ppm)								Alkalinity (pH)
		Total Dissolved Solids	Ca	Mg	Na	CO_3	HCO_3	SO_4	Cl	
308	Low alkali	1016	119	39	106	0	97	1143	113	6.3
351	High alklai	26905	153	13	1746	2432	3094	4210	996	11.4
402	High alkali plus NaOH	36044	113	73	12236	13230	3641	2513	359	11.8
188	High and low alkali plus $CaCl_2$	41366	346	50	12194	1417	3029	5370	14755	10.4

similar contamination. The degree of contamination will depend on how frequently the water may be changed.

It appears evident from the above data that direct contact between bars might have a considerable influence on comparative expansions by diffusion of alkalies. This supposition is substantiated by successive expansion readings on companion concrete bars which are stored in sets of three, one above the other, on racks in humid cabinets. The middle bar remains in the center position throughout the test but the top and bottom bars are alternated in position at each monthly reading. Typical results show that the bars expand materially more in the bottom than in the top position. See Table 2.

It has been observed that the contaminated water in the bottom of the mortar bar containers actually comes in contact with the bottom ends of the specimens in several ways; by sloshing as the containers are handled, by capillary movement along supports, and by diffusion with condensed moisture as it drains down and from the specimens. Bars thus become affected and start expanding entirely independently of any inherent reactive tendencies within the bars themselves. An affected

TABLE 2—EXPANSION OF BARS ACCORDING TO POSITION IN CABINETS

	Expansion in millionths of an inch per inch						
Month	Bar A Top Position	Bar C Bottom Position	Bar B Center Position	Month	Bar A Bottom Position	Bar C Top Position	Bar B Center Position
2	14	28	17	1	1	4	2
4	8	76	215	3	337	194	244
6	54	68	40	5	77	29	34
8	6	63	46	7	129	63	160
10	46	281	121	9	134	46	73
12	21	80	27	11	53	20	44
Total	149	596	466	Total	731	356	557

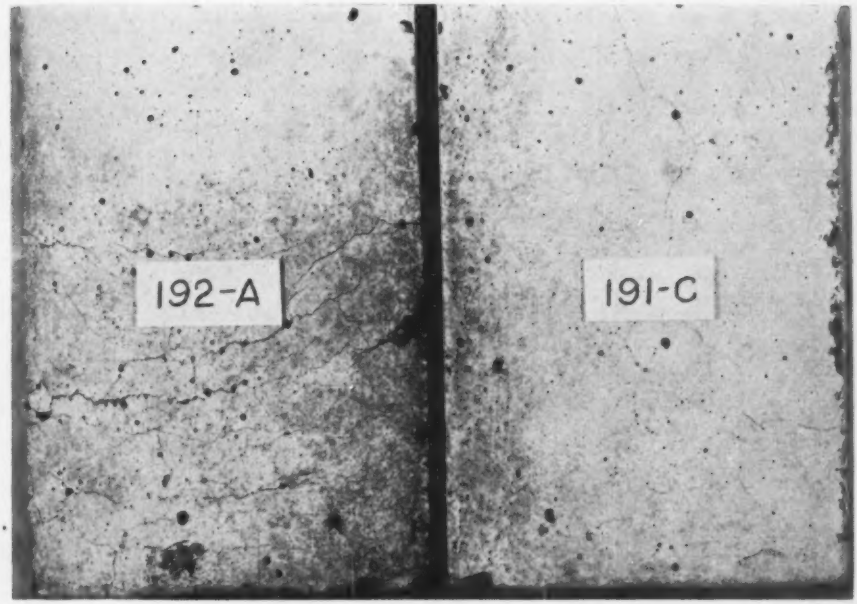


Fig. 18—A low-alkali bar (192-A) starts expanding at the lower end from external contamination while a high-alkali bar (191-C) expands uniformly from normal alkali-aggregate reaction.

bar (192-A) made with a low-alkali cement ($Na_2O + 0.658\ K_2O = 0.13$ percent) is illustrated in Fig. 18 in comparison with a companion bar (191-C) made with a high-alkali cement ($Na_2O + 0.658\ K_2O = 1.38$ per cent). Bar 192-A eventually cracked and expanded only in the lower portion whereas bar 191-C cracked and expanded uniformly throughout its full length. Bars affected by contamination do not always develop such pronounced visible disintegration.

Expansion and other data for the two sets of 1:2 mortar bars illustrated in Fig. 18 are given in Table 3.

CLOSING COMMENTS

Investigators have resorted to every practical expedient for accelerating the expansion bar test in order to provide a suitable method for predetermining troublesome combinations of cements and aggregates. At the same time, attempts have been made to utilize the test for comparing nearly every conceivable variable factor, including service characteristics, cements, aggregate types, grain size, and quantity of reactive constituents. The test was not devised for such comparisons and can be used for such purposes only with extreme caution and within very carefully established limitations, as has been amply demonstrated. The very

TABLE 3—EXPANSION, SONIC MODULUS AND WEIGHT CHANGE OF MORTAR-BARS

Admixture	2% $CaCl_2$ by Weight of Cement					
Reactive Agg.	19% of Size 100 Siliceous Magnesian Limestone					
Cement	1.42% Total Alkali			0.18% Total Alkali		
Test	Percent Expansion	Sonic Modulus	Weight Change	Percent Expansion	Sonic Modulus	Weight Change
Initial	—	4.51	—	—	4.53	—
1 Month	0.997	1.60	+0.74%	0.001	5.21	+0.09%
2 "	*1.040	1.90	1.11%	0.001	4.99	0.27%
3 "	1.060	2.05	1.39%	0.002	5.56	0.46%
4 "	1.058	2.41	1.48%	0.003	5.68	0.46%
5 "	1.054	2.38	1.48%	0.006	5.64	0.46%
6 "	1.067	1.54	1.66%	0.008	5.50	0.46%
7 "	1.067	2.94	1.66%	0.005	5.70	0.46%
8 "	1.071	3.16	1.94%	0.005	5.63	0.55%
9 "	1.075	3.33	2.03%	0.011	5.65	0.55%
10 "	1.073	3.37	2.13%	0.019	5.62	0.55%
11 "	1.074	3.42	2.13%	0.064	5.49	0.55%
12 "	1.075	3.48	2.13%	0.109	5.66	0.73%
14 "	1.076	3.42	2.13%	0.153	5.47	0.73%
16 "	1.078	3.54	2.13%	**0.266	5.06	0.82%
18 "	1.077	3.43	2.03%	0 357	4.65	0.73%

*Fine cracks appearing on all surfaces.
**Bottom 2 in. of bars disintegrating.

factors of acceleration such as shorter specimens, elevated temperatures, optimum quantities of highly reactive minerals, rich mixes and the closest approach to ideal moisture and storage conditions devisable preclude the use of test results for the purpose of indiscriminate comparisons involving indefinite periods of time. Extraneous influences may overshadow or completely eclipse the true effects of the particular factors for which comparisons are sought.

As an example of misapplication of expansion test data; it has been suggested that concrete made with a reactive aggregate and cement containing less than 0.5 or 0.6 percent alkalies will ultimately cause trouble in service because such combinations eventually cause excessive expansions in a highly accelerated mortar bar test. Such conclusions have been drawn in spite of the fact that it also has been repeatedly demonstrated by structures in service that concrete made with reactive aggregates and high alkali cement (containing appreciably more than 0.6 percent alkalies) develops expansive deterioration but that the same aggregate in combination with cements of moderate alkali content causes no trouble or at least results in materially improved service qualities.

Parker, Gene Wash, and Copper Basin dams were completed late in 1937 or early in 1938. These structures were all built with the same

aggregates (a natural gravel deposit at the confluence of the Bill Williams and Colorado rivers), and cement from the same mill containing 1.44 percent total alkalies. All three of these structures evidence profuse random pattern cracking and expansive deterioration, which started two or three years after completion.

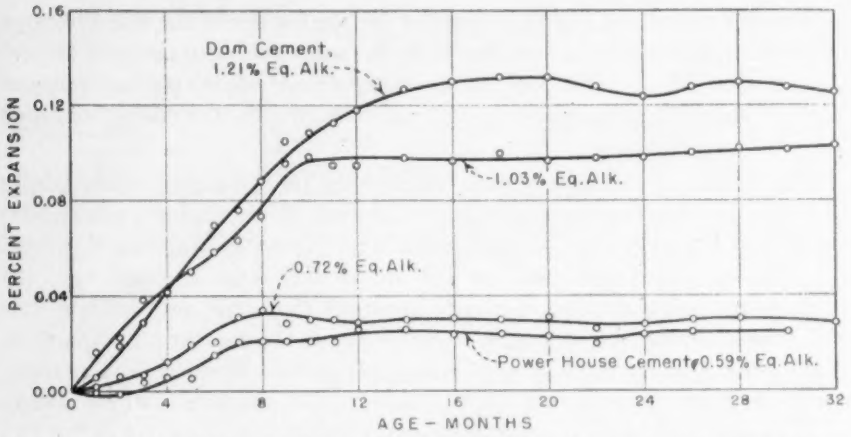
The Gene and Intake pumping plants were built almost coincidentally with the dams, using aggregates from Colorado River deposits a few miles below the mouth of the Bill Williams River. Investigations have proved that the two aggregate sources (1) are of the same geologic age, (2) contain the same mineral elements in about the same proportions and (3) show about the same degree of reactivity in the mortar bar expansion test. The cement used in the pumping plants contained 0.51 percent total alkalies. After 5 or 6 years of service the concrete in these plants is as nearly perfect as any concrete could be.

The same mill which supplied cement for the dams listed above also furnished cement for the Parker Powerhouse but with a specification provision which limited the alkali content to 0.60 percent of ($Na_2O+0.685$ K_2O). Actually the cement produced for the powerhouse contained 0.49 percent equivalent alkalies. Expansion test data for the Parker Dam aggregate in combination with the dam cement, the powerhouse cement, and two other cements of intermediate alkali contents are shown in Fig. 19.

Similar data and experiences have been reported or are now a matter of record for many other cases in California, Washington, Idaho, Nebraska, and Kansas. Thus it becomes evident that expansion test results, correctly interpreted within proper limits, can in reality be correlated with service performance.

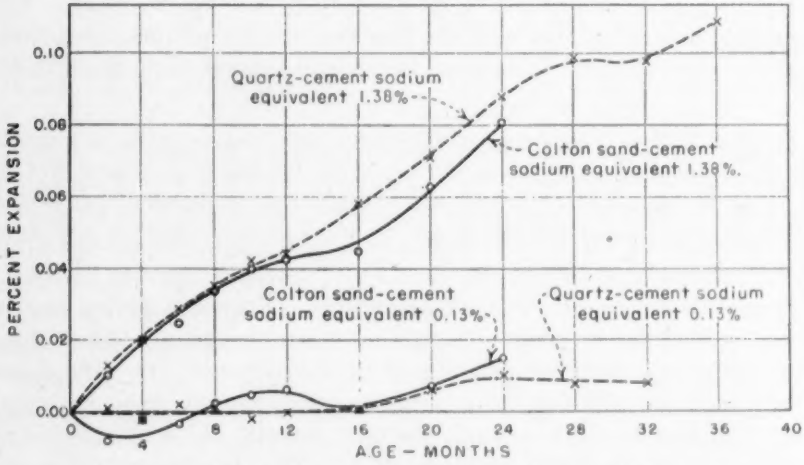
It has been suggested that all aggregates are to some degree reactive in concrete, with the possible exception of pure limestone. That this is true has been partially substantiated by the fact that excessive expansions eventually develop in a highly accelerated test, even with pure quartz aggregate and many other sands, gravels, and rock which have long records of satisfactory service (see Fig. 20). It is incompatible with sound engineering judgment to consider that the trouble which might develop from adverse alkali-aggregate reaction could be other than a matter of degree; that is, the lower the alkali content of the cement used with a reactive aggregate, the less severe the deterioration. A saving of even a few years of heavy maintenance costs will far more than repay the cost of any reasonable reduction in the alkali content of the cement.

The Bureau of Reclamation has now purchased several million barrels of cement under specifications limiting the alkali content. This cement has been produced by a number of mills, and there is unanimous agree-



1:3 mix. 1-by 1-by 10 inch bars. Bill Williams sand. Cement as shown. Bars stored sealed moist at 100 F.

Fig. 19—Parker dam aggregate produces expansion in proportion to the alkali content of the cement.



1:2 mortar. 2-by 2-by 10-inch bars. Stored sealed moist at 100 F.

Fig. 20—Some aggregates with satisfactory service records show excessive expansion in the mortar bar tests with high alkali cements.

ment that the result has been a much improved product. In most cases, the reduction in alkali content has been accompanied by marked reductions in C_4AF, somewhat lower C_3A, and increases in C_3S plus C_2S of as much as 9 percent. Depending on individual mill conditions, the additional cost for low alkali cement may range from nothing to about 20 cents per barrel.

The counsel of wisdom points unquestionably to the desirability of promptly adopting a reasonable limitation on the alkali content of portland cement for all cases where there is any possibility that the aggregate contain reactive constituents, in the meantime intensifying efforts toward a more satisfactory and complete solution of the complicated alkali-aggregate problem. Until a more satisfactory method is found, it is recommended that the reactive potentiality of aggregates proposed for use in concrete be appraised by means of a carefully conducted expansion test, using a finely ground cement containing one percent, or more, of combined soda and potassa.

Simple expansion specimens, 1- by 1-in. bars with an effective gage length of 10 inches, can be fabricated using molds, gage points and measuring equipment specified by The American Society for Testing Materials for determining the autoclave soundness of portland cement. These should be made, using a mix of 1:2.25 by weight of cement and sand with sufficient water to give a flow of 100 ± 5 according to A.S.T.M. Designation C87-42. The grading of the sand should be the grading intended to be used in concrete. The coarse aggregate can be tested for activity with high alkali cement by crushing to sand size, grading it to agree approximately with the sand grading except that not more than five percent of minus 100 mesh crushed material be used. After 24 hours moist curing in the molds, the specimens should be removed and their initial lengths taken. They should then be placed in sealed containers, which contain free water not in contact with the specimens, and these containers maintained at a constant temperature of 100 F. Provisions should be made for supporting the bars vertically in the containers, with the lower end of the bar (including the gage point) entirely above the water and with spacers to avoid contact between specimens. At monthly intervals the specimens should be taken from the containers and their length changes determined. Aggregates which show no more than 0.02 percent expansion after six months storage may be considered to be non-reactive, those which show expansions exceeding 0.04 percent in this same period or earlier should be regarded with suspicion, and any giving expansions of 0.1 percent should be considered definitely reactive.

[To facilitate selective distribution, **separate prints** of this title (42-21) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Aug. 15, 1946.]

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Vol. 17 No. 5 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **April 1946**

Concrete at Advance Bases*

BY I. S. RASMUSSON†

SYNOPSIS

Most of the advance base concrete work of the U. S. Navy in the Pacific Islands was done with coral aggregate because it was usually the only aggregate available. Producing a concrete generally inferior to that made with crushed rock or sand and gravel aggregates, coral served its purpose admirably in the temporary construction required in advance base planning. With proper selection and grading some corals will yield concrete with strengths little below that made of sand and gravel or crushed rock.

INTRODUCTION

Most of the concrete poured at advance bases in the South Pacific during the war was composed of cement and coral aggregate, bank run and graded. This was largely due to the fact that except on such large islands as New Guinea and the islands of the Philippines there was nothing but coral available. Even on the large islands, limestone and other hard rock comparable to that crushed for aggregate in the States could be obtained only with difficulty. So it was most natural to reach out and pick up the material which was plentiful and close at hand when concrete was to be mixed. This was invariably coral. On Guam, Luzon, New Guinea and on several other large islands, crushers and screening plants were set up in the hills to process volcanic rock. But, all in all, most of the concrete throughout the Pacific Islands was made with coral.

Concrete construction at advance bases was pretty generally confined to foundations, footings, paving, seaplane landing ramps, seawalls and a few dams. There were few multi-storied beam and girder or flat slab buildings; no arches, high dams, subways, nor vast reservoirs; concrete decks (Navy for floors) were poured under huts and warehouses; many

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walks were laid and in some places revetment floors and hardstands were built of cement-stabilized coral. Many of the huts and other buildings had footings and low foundation walls of concrete. There was considerable pre-cast concrete pipe molded and placed and a fair number of manholes, trench linings and small storage basins were constructed of concrete.

Construction methods were about the same as in the States. Most of the yardage was mixed at the site using the conventional 2-sack mixer loaded by hand from stock piles, although there were some instances where concrete was mixed in transit. When this was done, a 2-sack mixer with its traction wheels removed was mounted on a cargo truck and used to carry two batches, one revolving in the drum and the other ready in the loading skip. In the latter stage of the war, regular transit-mix trucks were used. If form lumber was not available, anything at hand was used—rough planking, plywood from old packing cases, even corrugated steel sheeting and pierced steel plank. If there were no sawn timbers for studding, wales, posts and bracing; logs were used.

CORAL

Before the war we knew little or nothing about coral concrete. Now, nearly everyone who was with the Seabees is, to some extent, familiar with its properties and uses. Reefs (Fig. 1) and deposits of coral in various forms are found throughout the tropical and sub-tropical islands of the Pacific, in Florida and the Caribbean areas. In Florida, coral is known as *oojua rock* and is approved for use in concrete road slabs and for precast piles for waterfront construction. Coral from reefs and deposits has been widely used during the last four or five years as aggregate for concrete construction purposes in the 14th Naval District.

The deposits in the Pacific Islands vary greatly in physical properties and in origin. They range from loose deposits in lagoons to ancient coraliferous limestones several hundred feet above the sea level (Fig. 2). Their bonding properties differ greatly according to the admixture of volcanic impurities, the proportions of coarse to fines and the percentage of organic material. Climate, traffic, sprinkling and the method of rolling also effect the rate of cementation when they are used for roads or runways without the admixture of cement. The war has made the United States conscious of the vast coral deposits in the Pacific for no other rock is available in the large group of islands such as the Lines, Gilberts, Marshalls and the Leeward Islands of the Hawaiian Archipelago. The presence or absence of coral deposits with bonding qualities, often determined whether or not an island could become an air base.

Coral, to the scientist, is the external calcareous skeleton of the coral polyp, an animal whose habitat is in ocean water, usually less than 100

Fig. 1. Fringing coral reef on Guam. Note terraces which range from a few inches to a foot or more in height.

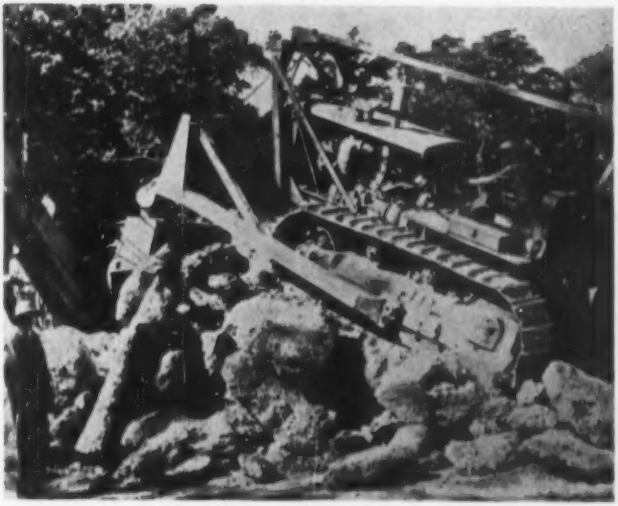

Fig. 2. Hard coral deposit several hundred feet above sea level at Espiritu Santo. This deposit is loosened by rooters and by blasting and the larger rocks are crushed by heavy tractors.

feet deep. It grows in colonies which develop various forms popularly known as staghorn coral, disc coral, mushroom coral, brain coral, etc. The skeleton is easily recognized by tiny holes with stellate partitions in which the polyp lived (Fig. 3). The skeleton, covered with these living animals, is live coral. When the polyps are absent the coral is dead. But to men building airstrips and pouring concrete in the South Pacific, the terms "live" and "dead" do not have this significance. Any white marine deposit that sets up well is called "live" coral, and any that does not, is called "dead" coral. Very little live coral, in the scientific sense, has been used in construction work.

The coral deposits of the Pacific are tremendously varied but fall into three main types: reef, beach and lagoon.

These may exist well above the sea level as compact limestones deposited when the ocean was higher, or as loose or well cemented deposits

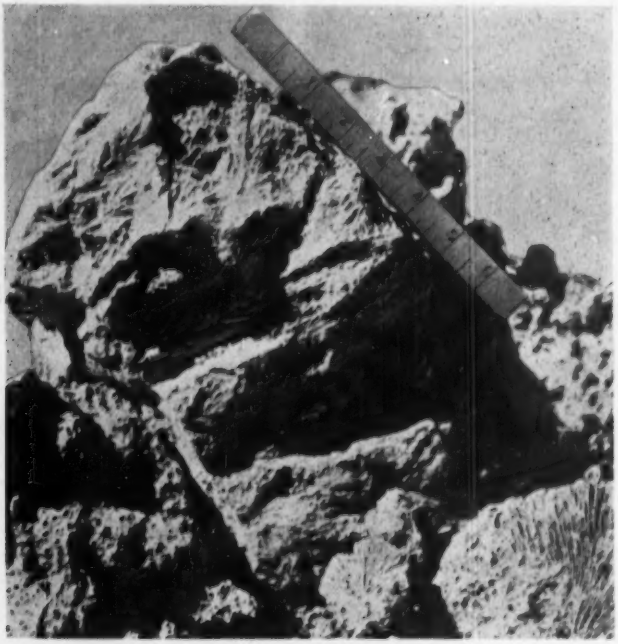

Fig. 3. A typical coral head showing tiny holes with stellate partitions in which the polyp lived.

near or below sea level. All three types have been used, ranging from sediments dredged from the Palmyra Lagoon in the Line group to hard limestone blasted and crushed from ancient reefs 450 feet above sea level on Guam in the Marianas and on Espiritu Santo in the New Hebrides. In general, emerged deposits have been used in the Tongan, New Hebrides and Solomon Islands, and submerged deposits in the Gilbert, Marshall and Caroline Islands. It is obvious that whether coral is dredged from the sea or blasted from the land, whether it is a few years old or half a million, does not determine its suitability for construction purposes. Coral deposits worked by the armed forces in widely separated islands had only three common properties: (1) marine origin, (2) high lime content and (3) white or cream color.

Dead coral deposits forming fringing reefs and lagoon fills in coral atolls are the common "live coral" of the Seabees in the Gilbert and Marshall groups. These are commonly dug up by a shovel or drag line into a causeway. The causeway is then trucked away progressively from the seaward end. It is normally used in bank run, saturated with sea water, and is easily handled because it does not require blasting and is usually the best natural mixture for bonding quickly in runways and roads.

CORAL CONCRETE

The following information covers the results of extensive study and tests carried on in the Pearl Harbor area.

Some hard reef corals, which have been deposited in layers, or "lenses," closely approach limestone in hardness and density. Specific gravities have been found to range from 2.40 for hard reef coral to 1.80 for finger coral. In making coral concrete, it is desirable to proportion the ingredients by weight, but on some advance bases it was impossible to determine the speficic gravity and absorptive characteristics of the local material. In such cases, the bank run coral was used in concrete, proportioned by volume, and adjusted by trial and error.

Most coral aggregate, stock piled and used on coral reefs, is produced by dragline or power shovel operations, and by dredging. All of these methods break up the consolidated aggregate and the material obtained exceeds the normal ratio of fine to coarse material. This is desirable, since laboratory investigations have shown that greater amounts of fine material are required for workable concrete mixes when using coral than when using normal concrete aggregates.

Classes of Coral Aggregate

Careful examination of an average washed sample of coral from a stock pile will disclose that the material may be roughly divided into two classes. (1) Sharply angular and fairly dense rock pieces of hard reef coral, which often resemble pieces of limestone. This material makes good general purpose concrete. It comes from old, dense, formations that have been stabilized by natural processes over a long period. (2) Elongated, finger-like specimens, that are rather light, fragil, and porous. This is known as finger coral, and is of comparatively recent geologic origin. It is light in weight and generally requires an admixture of coral sand to produce a graded aggregate. Finger coral makes an inferior concrete that is low in strength, difficult to place and likely to result in honeycombed structures.

Since both classes of coral are usually found mixed in various proportions, it is common to designate a coral as Class 1 or Class 2, according to whether hard reef or finger coral predominates. However, if the specific gravities can be ascertained, these classifications may be made more definite. In general, material ranging in weight from 75 to 90 lb. per cu. ft., loose volume, is placed in Class 1, while material whose weight ranges from 65 to 75 lb. per cu. ft., falls into Class 2. At times, finger coral may be found that will rate Class 1, especially if it has been processed through an aggregate plant. However, finger coral should not be depended upon to make first class concrete, unless it has been determined by preliminary tests that the material is exceptionally good.

Coral may sometimes be found that will sample over 80 per cent, passing a No. 4 sieve after sizes greater than 1½ in. have been removed. Such material is not recommended for practical use, since mixes produced with it will be closer to grout than concrete. On the other hand, material may be found of which only 30 per cent or even less passes a No. 4 sieve. Here there are not enough fines, and concrete made with this aggregate would be very hard to place and would be full of honeycomb. The amount of material passing a No. 4 screen in an aggregate, should be about 50 per cent. Such an aggregate would be ideal for concrete, giving excellent workability. Its strength, of course, would depend mainly on the quality or "class" of the coral.

Concrete having a maximum strength of 4,000 psi in 28 days can be produced with Class 1 aggregate. If Class 2 material is used, the strength of the concrete will rarely be better than half that figure. Aggregate produced by dredging operations usually contains a preponderance of well graded, fine material, while that obtained by dragline and shovel generally contains well graded coarse aggregate mixed with poorly graded beach sand.

In the procedure outlined here, it will be assumed that it is not practicable to separate the fines from the coarse aggregate; that the water can be measured in gallons; that the cement will be in bags of 1 cu. ft.; and that trial batches will be run to determine whether the bank run material contains the proper mixture of fine and coarse aggregate for producing a plastic concrete free from honeycombing tendencies. Pieces of aggregate larger than 2 in. in diameter will be screened out. This may be accomplished by making a rough screen of light reinforcing bars welded to form meshes 1½ in. square and passing all the material to be used through this screen.

Design of Mixes

The data in Table 1 are intended for reference and as a guide when designing coral concrete mixes. The figures in the table cover four

TABLE 1—BANK RUN CORAL CONCRETE—VOLUMETRIC MIXES

FINES IN AGGREGATE %	MIX	BAGS / YD	WATER - CEMENT RATIO	GAL / BAG	SLUMP	STRENGTH 7-DAY	STRENGTH 28-DAY
	1 : 3	10.43	.354	4.0	3 1/2"	3855	3937
	1 : 4	8.36	.425	4.8	2 3/4"	2910	3440
81	1 : 5	6.92	.521	5.88	3 1/2"	2244	2799
	1 : 6	5.95	.595	6.71	3"	1724	2136
	1 : 3	8.97	.543	6.12	3"	3337	3935
	1 : 4	7.06	.702	7.92	3"	3043	3419
30	1 : 5	5.85	.851	9.60	4 1/2"	2015	2573
	1 : 6	4.98	1.042	11.76	3 1/2"	1766	2408
	1 : 3	9.22	.468	5.28	3"	4152	4250
	1 : 4	7.45	.543	6.12	3 1/4"	3740	3955
50	1 : 5	6.19	.649	7.31	3"	2906	3274
	1 : 6	4.91	1.010	11.39	2 1/4"	2129	2448

different mixes of each of three types of aggregate. It will be noted that the slump obtained was in no case greater than 4 in., and that the average slump was about 3½ in.

The strength figures represent the average of three specimens tested.

Coral concrete is much more sensitive to over wetting than is concrete made of the ordinary aggregates, and its strength is greatly reduced by the addition of only a little too much water. The amount of water to be used per sack of cement was determined by trial, and where facilities and/or time were not available for making absorption and moisture tests on the aggregate, the slump was the sole criterion. If the aggregate is thoroughly dry, more water than usual will be needed, since the coral may absorb up to 100 per cent water by weight. Ideal coral concrete should, when discharged from the mixer, heap up in a cone in the barrow or buggy and show very little surface water. Such concrete will normally produce a 2 to 4 in. slump.

Trial batches of the mix decided upon should be placed in any convenient type of form and vibrated during placement, or tamped and spaded thoroughly if no vibrator is available. The forms should be removed in 24 hours, and if the concrete is honeycombed, indicating undersanding, beach sand should be added to the aggregate and further trials made until satisfactory results are obtained.

Some coral deposits have an excessive proportion of fine aggregates. A limit of 50 per cent of sand (material passing a $\frac{3}{16}$-in. screen) appears to be the maximum for good results. However, an aggregate containing as much as 60 per cent of fines may be used successfully by increasing the cement factor, which increases the amount of cement paste in the mix without disturbing the given water cement ratio. In this way, the workability of the concrete will be increased without reducing the strength.

At some bases further out, we did not discard material because it contained too much fine or too much coarse material. We used what we had or did not make concrete, as we had no time to look for other sources or to manufacture our own aggregate to specifications.

Much of the coral concrete was made with aggregate produced by passing dredged or excavated coral through an aggregate plant, crushing the larger pieces and then separating the coarse and fine aggregates and screening both to various sizes. Designed aggregates were then proportioned and mixed in a manner similar to that used with normal aggregates. By using the fine and coarse aggregates from a crusher plant, determining the specific gravity, moisture content, and absorptive properties of the coral, and then batch-weighing the fine and coarse material, concrete mixes of high strength and uniformity may be obtained. In Table 2

TABLE 2—STRENGTH OF CORAL CONCRETE

LOCATION	MIX AND BAGS PER/YD	SP GR OF CORAL	28-DAY COMPRESSIVE STRENGTHS		LOSS BY ABRASION (%)
			NATURAL (GRADED)	CRUSHED (GRADED)	
ISLAND X	E-1 (6.3)	2.32	3367		13.28
	F-1 (7.5)	2.15	3588	4224	4.90
ISLAND Y	D-1 (5.4)	2.20	3309		
	E-1 (6.3)	2.20	4194		
	F-1 (7.7)	2.20	4775		8.56
	F-1 (8.0)	2.20	5032		6.34
	F-1 (8.5)	2.20		5915	
ISLAND Z	D-1 (5.4)	2.19	3486		
	E-1 (6.3)	2.19	4551	4697	12.02
	F-1 (7.5)	2.19	5233	5441	7.24
OAHU (FINGER CORAL)	D-1 (5.2)	1.77	1650		
	E-1 (6.0)	1.77	1966		8.46
	F-1 (7.0)	1.77	2614		

are a few examples of the results obtained with aggregates from various sources in the Pearl Harbor area.

These mixes were all batched by weight, and were computed in accordance with Bureau of Yards and Docks standard specification 13Yc. Fines did not exceed 50 per cent of the total aggregate. In the mixes, aggregate containing only 45 per cent was used, and workable, plastic concrete having a low slump was produced. The slump should in no case exceed 4 in. (A slump of 3 in. or less, indicates a concrete that will give excellent results, with little bleeding).

Curing

To obtain the estimated strength and durability, coral concrete must be cured for at least 7 days by standard methods.

Forms can be removed from coral concrete structures in about the same time as from ordinary concrete.

Findings Based on CB Experience

Practically all of the construction battalions and maintenance units located in the Central and South Pacific have had occasion to use coral concrete. Their opinions of its worth and workability, how it should be mixed, and how it compares with concrete made of other aggregates seem to be in close agreement. The following notes represent findings based on the CB's experience in building coral concrete floors, recreation areas, walks, machinery foundations, tank cradles, culverts, etc.

A mix satisfactory for walls, footings, and rough finished slabs consists of 6 bags of cement per cu. yd. of selected coral in which the proportion of coarse to fine approximates 2 to 1. Six gallons of water per bag of cement with this mix will produce a uniform plastic concrete which will yield a smooth wall finish if well spaded. For slabs, eight gallons of water per sack of cement should be used to get a concrete that is not too stiff to flow and one that will screed readily and permit finishing with a bull

float. For galley and mess hall floors which will be subjected to repeated flushings when in use, 8 bags of cement per cu. yd. of selected coral should be used and the slab finished with a steel trowel to give a hard, smooth surface.

Coral concrete should be mixed for at least ninety seconds. After heavy rains, when the coral is very wet, it will tend to stick to the loader and to the sides of the mixer hopper. Therefore suitable time allowance should be made for the delay in getting aggregate into the mixer. When mixed, coral concrete flows cleanly and homogeneously, and may be hauled long distances without objectionable separation of the aggregate. It has been found that some batches that had been in the trucks for as long as ninety minutes were still plastic and workable.

Due to the varying sizes of the ungraded aggregate, the first screeding of a coral concrete slab presents some difficulties. However, once the larger lumps of coral have been forced below the surface, subsequent screeding will produce a smooth surface without too much trouble. Bull floats may be used satisfactorily on large slabs, followed by hand floating or troweling, depending upon the surface texture desired. Coral concrete is rather slow-setting and gives the finishers ample time to carry on his work carefully. A skillful finisher has been known to have completed 1,500 sq. ft. of slab surface in one eight-hour day.

The usual run of coral concrete used in the Pacific is less than half as strong as concrete made of crushed stone and sand or pit run gravel. It scuffs easily and is not waterproof. Six inch slabs with areas as large as 20,000 sq. ft. have been built without expansion joints and nothing more objectionable than hair cracks have developed over a period of six months. Nails can be driven into it with ease, as in cinder concrete.

Coral concrete may be placed in any type of form or mold generally employed in construction. Expansion joints can be kept to a minimum since there is little variation in temperature in regions where coral is found. Reinforcing steel can be used effectively.

Uses

Foundations and cradles for boilers, motors, heavy machinery, tanks, culverts and sewer inverts; floors for shops, quarters, mess halls and galleys and outside paved areas designed for light traffic have been successfully constructed of coral concrete.

One interesting job was the construction of a permanent reinforced concrete seaplane ramp on Leyte Island by the 88th Battalion. Constructed of precast reinforced concrete slabs, the ramp is 60 ft. wide and 220 ft. long, running from the beach at a slope of 6 per cent to a point 8 ft. under water at low tide.

The slabs are 8 ft. x 15 ft. x 15 in. thick, reinforced near the top and

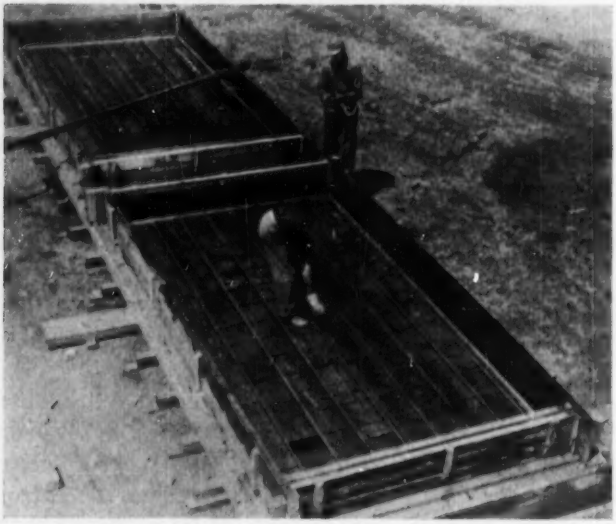

Fig. 4. Placing bottom reinforcing mat in slab form. During placing of concrete upper reinforcing mat will be set 1½ in. below top surface.

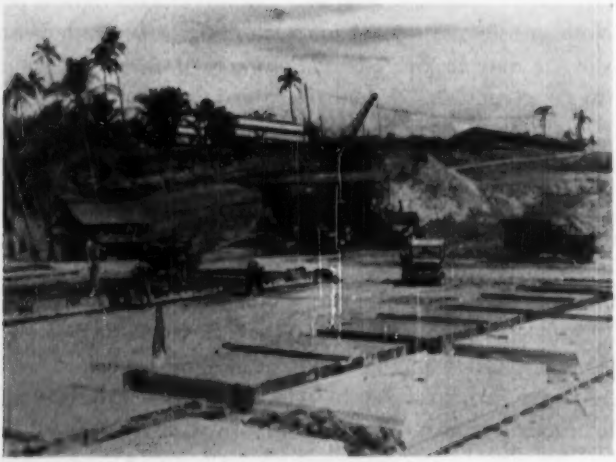

Fig. 5. Concrete slab factory. Mixing plant in background. Finished slabs curing in foreground. Note keys, keyways and 1 ft. sq. holes in finished slabs.

bottom by steel rods in both directions, tied together to form mats (Fig. 4). Keyways (Fig. 5) were formed in the ends of the slabs so that four of them could be joined together to form a full-width ramp section 8 ft. long. Supporting the slabs are five rows of 14 in. wide flange beams, spaced 15 ft. on centers and laid on a prepared subgrade. The ramp is anchored along both sides by dowels embedded in walls made of sacked lean concrete and the off shore toe is cut under the existing rock shelf to give a smooth approach. Practically all installation work was performed by divers. After the slabs were in place, crushed coral was forced pneumatically and also jetted through one-foot-square holes (Fig. 5) in the slabs until it completely surrounded the steel beams. This fill was then grouted through the same holes, giving the ramp a uniform, solid foundation. It has been giving complete satisfaction since it was built last summer and accommodates a 60 ton plane, having been designed for a wheel load of 45,000 lb.

CONCLUSION

All in all, coral concrete was used in thousands of structures of various kinds and was admirably suited for the temporary structures called for by advanced base planning.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

The Lorraine Viaduct, Switzerland

Concrete and Constructional Engineering, V. 40, No. 12 (Dec. 1945) pp. 259-260

Reviewed by GLENN MURPHY

This article describes a fixed-ended arch with a span of 492 ft. and a rise of 114 ft. 6 in., constructed over the River Aar by the Swiss Federal Railways. The arch is 44 ft. wide, 10½ ft. deep at the crown and 16 ft. 5 in. deep at the springings. It consists of top and bottom slabs connected by four longitudinal diaphragms. Design stresses of 27,000 psi for the steel and 1,420 psi for the concrete were used. The 28-day strength of the concrete was 5,690 psi.

Reinforced Concrete Chimney Base at Bradford

G. P. MANNING, *Concrete and Constructional Engineering*, V. 40, No. 12 (Dec. 1945) pp. 245-248.

Reviewed by GLENN MURPHY

A 350 ft. reinforced concrete chimney with a net inside diameter of 22 ft. has recently been completed at the Valley Power Station, Bradford. The lower portion of the chimney is designed to be used for office space and stores to a height of six floors. Movable forms were used in construction, an average lift of 5 inches per hour being attained. The walls were heavily reinforced to carry the vertical and wind loads from the chimney. The reinforcement was placed as concreting proceeded.

Precast Reinforced Concrete Cargo Barges

Concrete and Constructional Engineering, V. 40, No. 10, (Oct. 1945) pp. 206-211

Reviewed by GLENN MURPHY

The construction of 273 concrete barges was sponsored by the Ministry of Shipping beginning in 1939. The design adopted reduced the steel requirement from 56 tons to 18 tons for a 200-ton barge. Precast concrete construction was used throughout, the typical barge being composed of 126 precast beams and 42 precast ribs joined together with beams cast in situ. The basic mixture consisted of 11 cwt. of cement, 13½ cu. ft. of sand and 27 cu. ft. of ⅜ in. aggregate. Drawings and details of construction are included.

A Mobile Laboratory for Testing Concrete

Concrete and Constructional Engineering, V. 40, No. 12 (Dec. 1945) pp. 261-263

Reviewed by GLENN MURPHY

The Road Research Laboratory of the Department of Scientific and Industrial Research has developed a mobile laboratory for testing concrete. The laboratory, which

is built on a 6-ton trailer chassis, is equipped for performing standard tests on cement and aggregates, for determination of moisture content in aggregates, for measuring slump and preparing test cubes. It contains a 100-ton hand-operated hydraulic compression testing machine, sieve shakers, balances, molds and other equipment used in evaluating the quality of concrete. The laboratory carries its own power plant, but it may be connected to exterior sources of power. The cost is estimated as being between $5000 and $7000.

Multi-story Buildings of Precast Concrete

Mr. Donovan H. Lee, *Concrete and Constructional Engineering*, V. 40, No. 12 (Dec. 1945) pp. 255-258 Reviewed by Glenn Murphy

Describes a method of building construction developed by Mr. Donovan H. Lee, making use of prefabricated reinforced concrete members which can be used to construct any form of conventional building in the same way as structural steel or reinforced concrete. The precast reinforced concrete members are equipped with slotted steel connections which permit erection without bolts. They are designed to resist both shear and end moment. If desired, the beams may be cast with the top reinforcement exposed to permit the casting of a floor slab in place. The system has been evolved particularly to permit rapid erection of a fire-resisting frame at a low cost and with small dependence on weather conditions. The method is reported to be suitable for erection of buildings, rigid frame bridges, wharves, and water towers.

Compression Tests on Beams

C. H. Diggory, *Concrete and Constructional Engineering*, V. 40, No. 10, (Oct. 1945) pp. 202-205 Reviewed by Glenn Murphy

The author describes the use of test beams as a field method for evaluating the compressive strength of concrete. A series of beams 2¾ in. wide, 3½ in. deep, and containing 4 per cent steel were tested and compared with standard cubes. The beams, of 1:2:4 and 1:1½:3 mixtures, were designed to fail by compression in the concrete. Symmetrical loads, 30 in. from each end of a 78 in. span, were applied in 28 lb. increments. The load which produced a steadily increasing deflection was considered to be the crushing load. The modulus of rupture computed from the crushing load exceeded the cube strength of the corresponding concrete for each of the ten beams tested.

The author advocates the use of test beams for obtaining a rapid and straightforward evaluation of the strength of concrete on the job, and for eliminating the hazard of the transportation of specimens to a laboratory.

Prestressed Concrete

G. Magnel, *Concrete and Constructional Engineering*, V. 40, No. 12 (Dec. 1945) pp. 249-254 Reviewed by Glenn Murphy

Describes tests performed to measure creep and buckling. Three prestressed concrete test beams were loaded and their deformations compared with the deformations developed in control prisms. The creep of steel was measured both under conditions of constant load and constant length. Twenty-five test beams having a T-section 32 in. deep and 16 in. wide at the top were loaded on a 10 ft. span. The reinforcing in the beams consisted of twenty-four 3/16 in. wires fastened at the ends of the beam near the top and reaching to near the bottom of the beam at the center. The wires were prestressed to 120,000 psi. Beams which were designed for a concentrated load of 28 tons at the mid-span carried an ultimate load of 64 tons with failure occurring in the reinforcing wire. The 66 ft. beams built up from precast blocks with mortar joints were also constructed. The beams were designed for a working load of 7 tons at the midpoint and the one which was tested developed an ultimate load of 16.3 tons.

The torsional resistance of plastic materials with special reference to concrete

W. T. MARSHALL, *Concrete Constructional Engineering*, 1944, 39 (4), 83-8. *Building Science Abstracts*, Vol. XVIII (New Series), No. 1, January, 1945.

HIGHWAY RESEARCH ABSTRACTS

Circular and rectangular sections of a true plastic material under torsion were first investigated theoretically, and some experimental results obtained with similar sections of concrete in torsion were tabulated for comparison with these theoretical analyses. The ultimate torsional strength calculated on the assumption that concrete is plastic is nearer the determined tensile strength than the value obtained by assuming that the material is elastic. In many cases the small difference between the tensile strength and the plastic torsional strength may be attributed to experimental errors and to the usual anomalous behaviour of plain concrete under test. For different sections the ultimate strengths calculated on the assumption of plasticity are much more consistent (in one case they are all equal) than those calculated for an elastic material (values show a 25 per cent variation). Therefore, in torsion, concrete behaves at failure as a plastic material, so that the ultimate torsional moment of resistance may be obtained by taking the maximum stress as equal to the direct tensile stress of the concrete. Rectangular sections of concrete crack simultaneously along the perimeter and not at the mid-point of the longer side as would happen if the material were elastic.

Investigational Concrete Pavements—Progress Reports of Cooperative Research Projects on Joint Spacing

Highway Research Reports No. 3 B

HIGHWAY RESEARCH ABSTRACTS

A 108 paged printed publication which contains progress reports describing the condition of investigational concrete pavement projects built in either 1940 or 1941 in Kentucky, Michigan, Minnesota, California, Missouri and Oregon; and a report by the Public Roads Administration on "Structural Efficiency of Transverse Weakened Plane Joints."

Briefly, the experimental features common to the six State projects consist of a series of plain and reinforced concrete sections in which the joint spacing is varied. The plain concrete sections have transverse contraction joints at relatively close spacing (15 to 25 ft.) and expansion joints at 120, 400, 800 and 5281 ft. The reinforced sections have expansion joints at 120 ft. spacing with one intermediate contraction joint.

In general, load transfer devices were used in all expansion joints but were used in only part of the contraction joints of a given project in order to determine whether or not load transfer is needed with closely spaced contraction joints of the weakened-plane type. Several of the States included in their projects additional experimental features of design that were of particular interest to them.

During the three or four years since these pavements were constructed measurements and observations have been made of: (1) daily and seasonal variations in temperature, (2) daily, seasonal and progressive or permanent changes in the pavement, especially with respect to faulting at the joints, (4) the general condition of the pavement and joints.

Notes on controlling the uniformity of ready-mixed concrete

STANTON WALKER, *Technical Information Letter No. 23*, Aug. 22, 1945, National Ready Mixed Concrete Association. Reprinted *Concrete*, Vol. 53, No. 11, (Nov. 1945)

Reviewed by D. L. BLOEM and AUTHOR

Author points out that controlling the uniformity of ready mixed concrete involves attention to all details attendant on any concrete mixing operation, plus a few peculiar to the ready-mixed concrete procedure. Two classifications of ready-mixed concrete operations are discussed—central mixing and transit or truck mixing. A "combination"

procedure, called shrink mixing, is also mentioned. This involves partial mixing in the central mixer with mixing being completed in the truck mixer.

In central mixing no mixing water is required to be carried on the agitator—nor should any be permitted. Otherwise centralization of control is lost. However, if for any reason water must be added after the concrete has left the mixing plant it should be thoroughly incorporated by adequate mixing.

In transit mixing there are more things to be watched than for central mixing to assure control but it is emphasized that if all phases of the operation are properly controlled, uniformity as satisfactory as for any method of mixing can be attained. The problem of handling the measurement of mixing water is discussed in some detail. The difficulties of having precise knowledge as to the quantity of water in the aggregates is pointed out and it is suggested that control of consistency is an excellent method of controlling the quantity of the mixing water. An example is given with a table showing what variations in cement factor, water ratio and percentage of fine aggregate can result when the consistency of the concrete is maintained at a constant slump. It is pointed out that these variations are within the limits of the usual specifications.

The importance of water tight tanks and valves is emphasized. The problem of handling wash water is discussed.

Contribution to the Study of the Solution of the Lime of Portland Cements and Mixed Cements with a Portland Base

R. Zollinger, *Zement* 32, 17-18, 1943.
Digest in *Chemical and Metallurgical Engineering*, Vol. 52, No. 12; Dec. 1945.
Highway Research Abstracts

Presence of tricalcium silicate in portland cements leads to formation of a large quantity of lime during hydration. This basic compound has a tendency to combine chemically with compounds of an acid nature, thereby depriving the granules of cement of the protection given them by the colloidal calcium hydroxide formed by hydration, depending on the nature of the salt formed. The sensitivity to acids characteristic of portland cement is demonstrated in subterranean and underwater structures and in all cases where formation of $CaCO_3$, interferes with the above-mentioned colloidal type of protection. On the contrary, when substances having a good proportion of active silica are added, silicic acid can react with the lime liberated in hydration, yielding compounds which have a protecting action due to their colloidal character. Slags from high-temperature furnances, trass, etc. are ground together with portland cement yielding cements of lower sensitivity to acids than portland cement alone. The differentiation between protecting action of a physical-colloidal or chemical nature is shown by using different test pieces of ordinary German manufacture, either mixed with materials which contain active silica or alone. The test pieces used were extremely porous and were subjected to the action of pure water, saturated with carbon dioxide. The container has a capacity of 40 liters. The water is replaced as soon as its lime content exceeds 0.2 g. per 1. The diagonal position of the test piece is observed and the piece so arranged that most of its surface is exposed to the attacking substance. The test pieces are in the form of 1:5.5 mortars made with ordinary, inert sand. The possibility of secondary reactions through products which could be formed by corrosion of the vessel is eliminated by using glass. The test pieces were removed at intervals of from 3 to 210 days, and the losses in weight, variations in volume, lime content of the water, degree of corrosion of the test pieces and variations in resistance or flexion are determined. The tested cements are classified according to these characteristics, resulting in a widely differeing series. It was found that chemical protection is never sufficient to insure the resistance of the cement to the attacking substance, carbonic water. Of course there is a type of

chemical protection in which the age of the concrete plays an important role, since the protecting reaction which develops slowly increases the stability of the latter. In all cases in which the concrete is resistant to the attacking waters, the resistance must be attributed to the decrease in porosity by colloidal swelling of a physical type which adds its action to the insufficient protection of the physical type. According to this, the siliceous materials added to the cement should be judged more by their capacity for colloidal swelling by hydration than their equal reactivity with the $Ca(OH)_2$ liberated in the hydration of the cement.

Further studies of the bleeding of portland cement paste

HAROLD H. STEINOUR, *Bulletin 4 (1945)*, Research Laboratory of the Portland Cement Association, Chicago, Illinois. Reviewed by AUTHOR

In 1939 the Portland Cement Association issued Bulletin 2, by T. C. Powers, on "The Bleeding of Portland Cement Paste, Mortar and Concrete, Treated as a Special Case of Sedimentation." Bulletin 4 presents extensive further work that the Association has now done on the bleeding of cement paste. Subjects covered include the effects of water content, temperature, cement fineness and composition, and added chemical agents on both the rate and amount of bleeding. Duration of bleeding, and the degree of interdependence of rate and amount of bleeding are also considered. Principles that determine the rate of bleeding are reviewed in the light of special studies that were reported in detail in the Association's Bulletin 3. The details of the procedure for making the bleeding tests on the pastes are given in an appendix.

In general, the views on the mechanism of bleeding that were advanced in Bulletin 2 are supported by the later work. Certain points have, however, been further clarified, with some revision of theory. The equation for bleeding rate developed in Bulletin 2 now appears as a somewhat more approximate expression of the underlying principles than was at first thought. Nevertheless, it is considered to represent the experimental data for cement pastes with sufficient accuracy.

New equations for effect of water content on amount of bleeding, or bleeding capacity, are advanced as being applicable over wider ranges than the equation formerly developed. The new equations are empirical, since the effect of flocculation on the structure of the settled paste is not known in detail.

The earlier indications regarding the strong influence of the cement fineness on the bleeding characteristics have been confirmed; reduction in water content and increase in cement fineness are, in general, the most important ways of reducing the bleeding of normal, untreated portland cement paste.

The majority of the cement pastes were found to have about the same bleeding capacity at 90 F as at 74 F, and they changed in bleeding *rate* to about the extent that corresponds to the change in the viscosity of the water. However, the other cements were more strongly affected by change in temperature, perhaps because of changes in the amount of immediate chemical reaction.

The bleeding characteristics of a cement appear to be subject to as many influences as are other physical properties such as plasticity, setting time, and strength. Moreover, the rate and total amount of bleeding, though generally closely related, are not uniquely determined, the one by the other. It is entirely possible for cements to differ significantly with respect to the one property and not with respect to the other.

In a foreword, T. C. Powers, author of Bulletin 2, cautions that steps taken for the control of bleeding should be made only with adequate knowledge of their effects on all the important properties of the product. In general, differences in bleeding are regarded as secondary to other properties that must be controlled.

Shallow Patching adds Years to Life of Concrete Pavements

The Highway, V. 4, No. 5, Dec. 1945. (New Jersey State Highway Department)

HIGHWAY RESEARCH ABSTRACTS

It is sad but true that highways, like everything else, eventually wear out. It is this fact that keeps our maintenance division on the job 12 months a year, pouring joints, cold patching, mudjacking and performing a hundred and one associated tasks, all aimed at preserving the life of our highways.

One of the most interesting of these many operations is "shallow patching" of concrete road slabs. This patching, which varies in depth from "skin" patches of an inch or less, to patches of several inches in depth is actually a resurfacing operation. In New Jersey it has been used extensively wherever investigation shows that the concrete is solid below a certain depth and is damaged or "worn" out only on the upper portion.

This rejuvenation of damaged concrete riding surfaces is an old art with the maintenance division which did its first shallow patching over 18 years ago at a site along Route 4, where a burning gasoline truck had calcinated the pavement to a depth of 1½ inches. In this original operation (as well as in subsequent operations of this type) the damaged concrete was removed to insure a sound, clean surface. The perimeter of the area to be restored was hand chiseled to a vertical edge and the surface thoroughly cleaned by hand brushing with a wire brush. In this highly experimental patching of 1927, a very "rich" mix (1:1:2) was used, with four gallons of water added for each bag of cement. Immediately following the addition of 2½ per cent of calcium chloride, this "dry" concrete, which was hand mixed and allowed to approach its initial set before placement, was spread over the patch area and thoroughly tamped. It was then struck off to somewhat above the finished grade.

About fifteen minutes after the initial set, the mixture was thoroughly retamped, screeded and planed to the proper section. Curing was accomplished with damp earth and the road re-opened to traffic in 12 days. After 18 years this original shallow patch shows no marked indication of failure.

From this small beginning has evolved a number of refinements, although it cannot be said that later developments have produced better results.

Recently the maintenance division did some shallow patching on Route 25, just south of Route S-41. This patching was done on pavement that was originally laid in 1926 and covered an area of approximately 75 sq. yd. The depth of replacement on this job varied from 1½ in. to the full depth of 8 in. The mix, in this case, was 1:1¾:3½ and the water content 4½ gal. per bag. One per cent of calcium chloride was used.

Curing in this instance was accomplished with burlap and water and unlike the original patching, curing time was reduced to 4 days, due to the traffic demands on this particular highway. Another difference between this and the job of 18 years ago was the use of machine mixed concrete.

Prior to placement of patching concrete the cleaned surface of the old concrete was thoroughly wet and dusted with cement. Particular attention was paid to the placement of concrete immediately adjacent to the old edges in order to prevent any infiltration of surface water.

After being subjected to heavy traffic for two months, these latest patches show no failures. This is most encouraging since this job may well be classified as experimental inasmuch as it was impractical to keep traffic off the highway for a long period of curing.

Creep of Concrete at High Overload

J. R. SHANK, (Ohio State University) *Engineering Experiment Station News*, Oct. 1945
HIGHWAY RESEARCH ABSTRACTS

The stress-strain diagram for concrete, particularly when initially loaded, is usually not a straight line. Calculations based on the straight-line principle do not predict the ultimate strength of beams with any satisfactory degree of accuracy. Creep is a property of importance even in short periods immediately after loading. It therefore appears that strain in concrete is made up of two factors, the elastic and what may be called the plastic factors.

Determinations for plastic flow have been made largely on a long time basis and for loadings up to, but not much higher than, working loads. This gave data for calculating the redistribution of stresses in members and frames due to changes wrought by long-time plastic flow but which was not of much value for separating the two factors in the stress-strain curve, for evaluating the part that plastic flow takes in the upper ranges of loading, or for explaining the inadequacy of the straight-line principle in forecasting ultimate strengths in flexure. Knowledge of the creep or plastic flow in the upper loading brackets seems to be necessary for this solution.

A concrete was made up to conform to the 1-6½ mix of the Ohio Department of Highways specifications using gravel and sand.

Modulus of elasticity determinations were made in a manner that has been used for some time at The Ohio State University. The cylinder with strain gauges on opposite sides is first loaded at ordinary test loading speed to 70 per cent of the expected ultimate strength, released at the same speed to near zero loading, loaded similarly a second time, released a second time, and then loaded a third time when strain readings are taken.

Sufficient tests of this sort have been made at Ohio State through the years to prove that the curve is a straight line.

Other cylinders of the same concrete mix were made, stored, and tested for plastic flow over 24-hour periods. Loadings of several high percentages of the expected ultimate loading were applied.

Data obtained when plotted on log paper showed that formula such as

$$y = C \sqrt{\frac{a}{x}}$$

y = micro-inches per inch per one psi
C = a coefficient
a = a root or the reciprocal of a fractional exponent
x = days

could be fitted closely to them. Breaks in the straight lines which such a formula should produce indicated for some of them that changes had to be made in C and a at certain times.

The changes in the curve formula values were all in the direction of less plastic flow for the longer times. This is shown largely by the increase of the root figure and the decrease of the coefficient.

The indications are (1) that concretes that have been pre-worked and loaded so as to reduce the plastic flow properly by rapid loading may be considered elastic in the same sense as cast iron and other granular metallic concretes and (2) that creep or plastic flow increases in the upper loading stages to very high values at or near the breaking point. These tests afford hope that the elastic and plastic properties may be viewed separately and so make possible improved rational theories which are more satisfactory than the theories that ignore the plastic flow or those built purely on empirical data which clash with the engineer's sense of what actually takes place, however well they fit breaking test data.

Experiments on Reinforced Concrete Beams to Determine the Limiting Reinforcement (Versuche an Stahlbetonbalken Zur Bestimmung der Bewehrungsgrenze)

W. Gehler, H. Amos and E. Friedrich: *Deutscher Ausschuss fur Stahlbeton*, Heft 100. Berlin, 1943 (W. Ernst and Sohn). *Building Science Abstracts*, V. XVII (New Series) No. 12, Dec. 1944.

Highway Research Abstracts

A report is presented of an investigation of reinforced concrete beams which was carried out at the Testing Laboratory of the Technical College, Dresden, for the German Reinforced Concrete Committee (Deutscher Ausschuss fur Stahlbeton) during the years 1937 to 1939. The report consists of three parts: Part A is an introduction by Gehler in which the concepts and notation used are elucidated in relation to the hitherto usual methods of computation; the "bearing value" of a beam under a given degree of loading is defined; the two limiting values of the bearing value are equivalent to the strength T_s, when rupture takes place due to the steel yield point being exceeded, and T_b, that at which the concrete fails in compression-bending. The "limiting reinforcement" refers to the limiting case $T_b = T_s$ when the strengths in both the concrete and the steel zones are the same and both materials are fully utilized. The aim of the investigation is, in particular, to show that with special care in carrying out the strength tests this limiting reinforcement can be determined (or the zero-line coefficient at the limiting reinforcement) for beams made with different types of steel and quality of concrete; in this way a new material value can be obtained and after further study applied to practice. These exact experiments are intended also for determination of the ratio of the bending moment at which the first cracks appear to the moment of rupture. In Part B is given an account of the bending-compression tests of 188 beams of rectangular section with or without compression reinforcement. Reinforcing steels of 4 different yield points were used, namely steels St 37 and 52, the twisted bar steel known as "Drillwulststahl" and the special concrete steel of the Peine steelworks; the concrete was of two qualities, one having a 28-day cube strength of 120 kg. per sq. cm. (1704 lb. per sq. in.) and the other of 160 kg per sq. cm. (2272 lb. per sq. in.). The objects of the experiments, as stated in this section, were to determine the actual "limiting reinforcement" between lightly and heavily reinforced concrete cross-sections, to compare the concrete compressive stress at rupture with the design strength based on the cube strength of the concrete, and to study the bahaviour of compression reinforcement to discover to what extent compression bars as introduced over the supports of continuous beams influence strength and the limiting reinforcement. The results are shown in the form of diagrams and tabulated data, and photographs of the beams, etc., are reproduced. In Part C, Gehler examines and applies the results and elucidates the "Dresden" method of computation for the plastic concrete zone. The investigation, it is pointed out, is to be regarded as a contribution to fundamental research on reinforced concrete on the following grounds: The results allow conclusions to be drawn respecting the plastic behaviour of concrete in reinforced concrete beams; they confirm the correctness of the hitherto usual method of computation for lightly reinforced beams; they provide a basis for a new method of design for heavily reinforced beams in the plastic concrete compression zone; they allow simple expressions to be formulated for determining the limiting reinforcement for more extensive utilization of the elastic steel zone of lightly reinforced beams; they facilitate the work of computation in dimensioning and simplify construction by eliminating compression bars and diagonal reinforcement. Conclusions are reached also on the factors influencing freedom from cracking, and suggestions are made respecting the improvement of bond and freedom from cracking.

Chemical Processes in the Hardening of Portland Cement

RUNE HEDIN. Svenska Forskningsinstitutet for Cement Och Betong Vid Kungl. Tekniska Hogskolan I Stockholm. (Swedish Cement and Concrete Research Institute at the Royal Technical University Stockholm) 1945

HIGHWAY RESEARCH ABSTRACTS

The investigations were intended to throw light on the chemical reactions that take place when a mixture of portland cement and water hardens under normal conditions.

From the results the following conclusions have been drawn:

All the clinker materials are dissolved in the water while retaining their stoicheiometric compositions. The crystallized minerals have a somewhat lower rate of solubility than the minerals in the form of hardened "melt." The solution formed contains chiefly calcium-ions, silicate-ions and aluminate-ions. Owing to hydrolysis there are present also free silicic acid and aluminum-hydroxide in the solution. By dissolution of added substances mixed with the cement, e.g. gypsum, the ion concentrations and ion activities in the solution can be changed.

With a sufficiently high aluminate-ion concentration the free silicic acid is coagulated. At the same time the aluminium-hydroxide is precipitated by the silicate-ions. Through coagulation the concentration of the calcium-silicate in the solution is kept very low, and the dissolution is therefore extremely rapid. As the dissolution of the tri-calcium-silicate is a strongly exotherm reaction, the cement mass is exposed to considerable heat. The gel-products formed quickly enclose the cement grains, so that these are prevented from coming into contact with the water. The reaction thus stops after a relatively short time.

If the calcium-ion concentration in the solution, on account of the composition of the cement clinker, or by additions, is kept high in relation to the existing amount of aluminates, the concentration or activity of the aluminate-ions will be so far reduced that the above-mentioned coagulation-reaction does not take place. The silicates and aluminates present in the solution will then be hydrated to form compounds whose solubility is less than the solubility of the original anhydride materials. The fluid phase will thus be super-saturated the whole time relative to these products of hydration that crystallize; but it will be unsaturated relative to the anhydride material. Consequently, the reaction proceeds until either the water has been consumed or all the anhydride material has been converted.

The composition of the hydrated products is completely dependent on the concentration conditions obtaining in the solution.

The aluminates give rise to tri-calcium-aluminate-hydrate or tetra-calcium-aluminate-hydrate; or if gypsum is present, to tri-calcium-aluminate-mono-sulphate-hydrate.

The silicates are hydrated to calcium-silicate-hydrate, with a molecular relation $CaO:SiO_2$ that in saturated lime-solution is 3:2, but in the case of strong super-saturation of the solution with lime may rise to 2:1. As the silicate-containing anhydride material consists chiefly of tri-calcium-silicate, the lime-content of the solution will rise rapidly to strong super-saturation and calcium-hydroxide will crystalize. It has been possible to show the main component of the portland cement, alite, has a very much higher rate of solubility than has pure tri-calcium-silicate.

The ferrites have a very low rate of solubility, and therefore they exercise scarcely any influence on the reaction before the cement-mass has begun to harden to any extent. They are, however, gradually converted to ferri-hydroxide, which on account of its structure envelopes the cement-grains like a protective film and retards the dissolution. If the concentration of lime and gypsum in the solution is high, however, the rate of solubility of the iron components is near to zero, and they then have no noticeable effect on the other reactions. Moreover, the ferri-hydroxide that may in this case have been formed is converted to the crystallized compounds tetra-calcium-ferro-hydrate or tri-calcium-ferri-mono-sulphate-hydrate.

The alkalies in the portland cement that occur, for the most part, in the form of alkali-containing aluminates, have an extremely high rate of solubility, and they give rise to alkali-containing solutions, in which no high calcium-ion concentration can be attained. The silicic acid is thus coagulated even if free lime is present in excess. If there is excess of gypsum, however, a sufficiently high calcium-ion concentration can now be attained in the solution, and the coagulation reactions does not take place.

If a big excess of an easily soluble calcium salt such as $CaCl_2$ is added to a portland cement, the pH-value of the lime-solution formed is reduced to about 11. The aluminate-hydrates are more easily soluble in such a solution than the aluminium-hydroxide. The latter then crystallizes and binds the grains in the cement-mass together and false quick-setting results.

In the presence of borax, humus, sugar and other substances that give rise to calcium salts with a low degree of dissociation, or insoluble calcium compounds, the calcium-ion construction in the solution cannot attain a sufficiently high value and the consequence is a pronounced coagulation of silicic acid.

The 150 paged bulletin includes 64 tables, 32 figures and 79 references.

Action of steel in reinforced concrete pavement

W. R. Wooley, Materials Engineer, Chicago District, Public Roads Administration

Highway Research Abstracts

H. F. Clemmer, Engineer of Materials, District of Columbia, in the November 1944 issue of Highway Research Abstracts states: ". . . it is believed that reinforcement is of particular value in distributing shrinkage stresses, developed during hydration of the cement, which tends towards incipient cracks that may later develop into structural failures." We agree with the first part of this statement, i.e., that reinforcement is of value in distributing shrinkage stresses, but we believe that the last portion of the statement gives an incorrect inference. Mr. Clemmer's thought seems to be that shrinkage causes incipient cracks to form in plain concrete which later develop into visible cracks, whereas reinforcing steel prevents the formation of incipient cracks. We believe just the reverse is true. The steel mesh, because of bond and because of the welded cross wires, actually causes numerous cracks at short intervals while the concrete is setting, which are either incipient or too small to be seen under ordinary circumstances. It is our belief that such cracks do not form in plain concrete, but that any cracks that do occur, immediately open wide enough to be plainly visible.

This thought seems to be borne out by photographs taken shortly after a rain as a 9-year-old reinforced concrete pavement was drying. Moisture has indicated the existence of numerous fine transverse cracks, which are normally not visible. These cracks were invisible a few minutes after the photographs were taken and when the moisture had evaporated. It seems that the reinforcing steel breaks the concrete during the period of initial shrinkage into very short sections—in this case generally less than one foot long. The cracks are so small that they ordinarily escape detection.

Their effect is to distribute the shrinkage more or less uniformly throughout the length of each section of concrete. As shrinkage, contraction, warping stresses, etc. develop, the strain becomes too great to be absorbed by these microscopic cracks. Some slippage of the steel probably occurs adjacent to an occasional microscopic crack allowing it to open sufficiently to cause a visible crack. Because some of the strain is absorbed by the microscopic cracks, the visible cracks in reinforced concrete are generally farther apart than in plain concrete. Thus, in reinforced pavements having joints every 30 feet or less, as in the practice in the District of Columbia, no visible cracks occur. We believe this explains why cracks, as ordinarily conceived, are farther apart in reinforced concrete pavements than in plain concrete.

The amount of longitudinal steel required to prevent wide cracks in long reinforced concrete pavement slabs is a subject not clearly understood, and one which we do not propose to discuss here. We do believe, however, that the line of reasoning just outlined can be applied to reinforced concrete slabs to explain why considerably less steel is required than has generally been believed necessary, particularly for extremely long slabs.

Mr. Clemmer's reply—I know that oftentimes small cracks, or as I think of the condition, "hair cracks or crazing" occur in pavement surfaces and we have experienced this trouble in the District of Columbia; however, I have never associated this cracking with the use of reinforcement. I could realize the possibility in the case of large bars, but not where mesh of 50 pounds (No. 4 and 6 wire) or less is used and placed 3 to 4 inches, according to the thickness of the slab, below the surface of the concrete. We have had this trouble in plain concrete in alleys probably more often than when reinforcement is used. Certainly if these "hair cracks" are caused by the reinforcement it does reduce their width so that autogenous healing may take place if proper wet curing is provided while if reinforcement were not used, there might be fewer cracks but of such width as not to permit healing, and possibly developing into structural failure.

My belief is that this cracking may be the result of the combination of circumstances. We have found that some cements, though they pass the specified requirements for "time of set," do not take set in a very short time. If final finishing (floating) should take place at this time it is very possible to cause these fine surface cracks. I have known instances of the use of hot dry stone for aggregate, the trouble has been corrected by wetting the stone previous to mixing; however, the aggregates used in the District of Columbia are saturated sand and gravel. In general, this cracking has occurred on rapid drying days, that is, where the humidity may be low and a warm breeze is present; early wet curing has then corrected the trouble. On very hot days too early placement of wet burlap might cause such rapid cooling of the surface as to increase rather than to eliminate this crazing. My practice has been to study the conditions carefully and try various expedients.

As I stated, we experienced this hair cracking in recent years but have not considered the use of mesh reinforcement placed one-half the depth of the slab to have caused this trouble. It is believed that light weight (20-lb.) mesh placed 1-in. below the surface around all structures extending the full depth of the slab would assist in preventing this cracking. We have never found such cracking where this light weight mesh has been used.

ACI NEWS LETTER

Vol. 17 No. 5 JOURNAL of the AMERICAN CONCRETE INSTITUTE April 1946

A Review of the 42nd Annual ACI Convention

By FRED F. VAN ATTA

The first post-war ACI convention at the Hotel Statler in Buffalo, February 19, 20, 21 was a busy one and a good one. Registered attendance was 325 (174 members and 151 guests). Last year in New York we had 72 at the "token" convention (no technical program) and in 1944 the registration in Chicago was 376.

When it was found that New York might be too crowded to handle the convention as originally planned there was much scurrying around to find an alternate location. Chicago offered no better assurances. Everyone knows what Washington was like. The interval of indecision hindered progress on the program. When Buffalo was selected, the preparation period was short. Doubts about an ample program inspired extra efforts and more program commitments. The results broke long records for six full sessions—an unusually heavy, meaty, tight-packed schedule. Many said "the best program ever" and some said "an endurance test."

Major credit for the top notch meetings goes to Robert F. Blanks, Chairman of the Publications Committee, and his corps of "Session Leaders" and to Committee 115, Research.

One whole session was devoted to "Entrained Air in Concrete." The lead paper on laboratory studies (Feb. 1946 JOURNAL) by C. E. Wuerpel was followed by 12 ten-minute contributions on a variety of aspects of the subject.

There were six papers on repair and maintenance, three on reinforced concrete design, three on precast concrete structures and one each on "Radiant Heating by Reinforced Concrete," curing compounds, cement dispersion, Navy wartime construction in the Pacific and bridge pier construction by the "Prepakt" method. There was an illustrated talk on effects of the atomic bomb in Japan and one on observations made during a post-war trip to Germany last summer.

Two committee reports, (precast floor units and silo construction) were adopted as Institute Standards subject to ratification by letter ballot.

Three technical committees and one subcommittee held meetings during the convention.

Mr. Harrison F. Gonnerman was elected President of the Institute with other officers and directors.

Wednesday noon 186 ACI members and their friends sat down to a luncheon in the Fillmore Room. There was no program but the occasion offered an excellent opportunity to renew old acquaintances and make new ones in congenial surroundings over a good meal.

Thursday afternoon a group of about 35 made an inspection trip to the Niagara Falls Power Company Plant at Niagara Falls and visited the Falls on the American side. The huge masses of ice at the base of the falls, the ice encased trees, the recent heavy snow and the multi-hued green, blue and white water presented a beautiful scene quite unlike the one usually associated with "honeymooners mecca."

Pre-cast Concrete Floor Units, New Developments In Concrete Design Methods

Afternoon, February 19

Due to the short interval of convention program development, several important contributions had not been fully prepared for publication. For the same reason very few papers had had pre-convention publication. As the April Journal goes to the printer the publication schedules have not been completed. Papers will be published as soon as possible after they become available. Meanwhile ACI members and others will please be patient. It will not be possible to supply copies of unpublished convention contributions.

Taking its cue from a criticism expressed following the convention in 1944—that the program failed to provide material of special interest to the structural designer—the Publications Committee, under the Chairmanship of R. F. Blanks, planned this session for structural engineers. As a man who has broad knowledge in this field, A. J. Boase was selected as session leader and is to be commended for his fine work in securing contributions of interest and in leading the afternoon program.

President Parsons opened the session with the appointment of Jacob Fruchtbaum and Morgan B. Klock as tellers to count the ballots of the annual election. Before turning the session over to Mr. Boase he called on Professor Menefee to submit a committee report.

Precast concrete floor units

Chairman F. N. Menefee presented the report of Committee 711 "Proposed Minimum Standard Requirements for Precast Concrete Floor Units," by reference to its publication in the January 1946 JOURNAL. The convention approved the report for adoption as an ACI Standard, with one editorial revision subject to ratification by the Institute membership by letter ballot.

Radiant heating by reinforced concrete

This paper (p 513 this JOURNAL) presented by John R. Nichols, shows radiant heating to be a method which is rapidly becoming popular in the United States. Heating systems of this type, more precisely described as "panel heating," have been in successful operation in England and on the Continent for many years and many installations have been put in service in this country. There is great value and convenience in this type of heating and Mr. Nichols, who is primarily interested in the structural aspects of the problem, believes that we should scruti-

nize the building codes which prohibit the embedment of heating pipes in structural concrete to be sure that progress in this field is not needlessly impeded.

Many features of this type of heating were explained in a prepared discussion by P. B. Gordon, heating engineer of the firm of Wolff and Munier, Inc., New York. The temperature of heating pipes embeded in concrete never exceeds 180 F and the usual range is 100 to 140 F. Resulting stresses do not exceed those of the reinforcing steel in the concrete. Francis R. MacLeay, Corbetta Construction Company, New York, and Chester L. Post, Public Buildings Administration, Washington, contributed to the general discussion.

Hipped plate construction

Professor George Winter, Cornell University, author of this paper described a design method which utilized effectively the monolithic character of concrete, particularly applicable to roofs of moderate span, bins, etc., which has had little use in this country. Professor Winter has had occasion to design and see several substantial structures of this type erected during a number of years spent as a consultant in Russia. Stereoptican slides illustrated his design procedure and showed examples of several completed structures in Europe and one in the United States.

Anton Tedesko presented a prepared discussion. Additional discussion was contributed by Frederick S. Merrit, *Engineering News-Record*, and Mr. Post of the Public Buildings Administration.

The lattice analogy in concrete design

Presented by Douglas McHenry, Bureau of Reclamation, this paper describes a method whereby many complicated problems in stress distribution can be solved by a method involving only simple arithmetic. The field of application of the lattice analogy is comparable to that of the photoelastic polariscope, but the elaborate equipment of photoelasticity is replaced by the ordinary computing machine. Mr. McHenry made use of many slides illustrating the progrssive steps to be followed in the solution of problems by this method.

Prestressed concrete barge

Dr. Henri Marcus addressed the convention, using stereopticon illustrations of the design and construction of a concrete barge by the Navy using prestressed reinforcing steel. At its conclusion Session Leader Boase told of a prestressed model girder he and F. R. McMillan had examined while in Europe and of some 2.8 in. thick concrete floor slabs in South America that had been built using Dr. Marcus' method of design.

Entrained Air In Concrete

Evening, February 19

F. H. Jackson, who so ably handled the session on Air Entrainment at the 1944 Convention did another "bangup" job this year in marshalling through on schedule the program of one 40-minute paper and 12 "ten-minute" contributions on this interesting and timely subject. The lead contribution by C. E. Wuerpel was taken from the high points of his paper, "Laboratory Studies of Concrete Containing Air Entraining Admixtures" published in the February 1946 JOURNAL.

The short papers which made up the balance of the program were: "Recent Experiences with Air-Entraining Portland Cement Concrete in the Northeastern States," by Lee Andrews, "Entrained Air—a Factor in the Design of Concrete Mixes," by W. A. Cordon; "Experiences with Air-Entraining Cement in Central-Mixed Concrete," by Alexander Foster, Jr.; "Homogeneity of Air Entrained Concrete," by Henry L. Kennedy; "Effects of Use of Blended Cements on Durability of

Concrete," by W. F. Kellermann; "A Method for Direct Measurement of Entrained Air in Concrete," by W. H. Klein and Stanton Walker, presented by Mr. Walker; "Methods of Entraining Air in Concrete," by Dr. E. W. Scripture, Jr.; "Studies of Concrete Containing Air," by Stanton Walker and Delmar L. Bloem, presented by Mr. Bloem; "Automatic Dispensing Equipment for Air-Entraining Agents," by R. R. Kauffman; "A Mechanical Dispensing Device for Air-Entraining Agents," by E. M. Brickett; "A Simple, Accurate Method for Determining Entrained Air in Fresh Concrete," by S. W. Benham, read by Mr. Kellermann and "Air-Entraining Concrete," by W. H. Herman, read by Mr. Jackson. At the conclusion of the prepared discussions Mr. Wuerpel added some remarks on tests made on mass concrete with entrained air which indicated that advantages can be obtained here as well as in pavement work by the use of air-entraining agents.

B. H. Wait of Wait Associates contributed a discussion of the use of blended cements to obtain desired resistance to frost action, read by Robert Wait.

J. S. Spruiell and several of the contributors listed above participated in the general discussion.

Concrete Research

Morning, February 20

The annual open meeting of ACI Committee 115, Research, started promptly with Vice-Chairman Richart presiding. Attendance was about 175 and held well until adjournment for the luncheon. The program was packed and brisk, consisting of eleven short informal reports on research projects selected from among many items submitted by 55 reporting laboratories in reply to a questionnaire sent out by the Secretary of the committee, Prof. S. J. Chamberlain, Iowa State College, Ames, Iowa.

Two reports were presented on air-entrainment: one on how air-entrainment greatly improved the durability of concrete made with a troublesome fine aggregate when other methods failed; and the other on the relative amounts of air obtained with different air-entraining cements and different concrete mixtures. One speaker reported that on the few tests made to date the indications are that axially-loaded concrete can withstand only about 70 per cent of the ultimate strength for an indefinite period. A contributor from the floor reported that concrete structures subjected to very short time loadings from the atomic bomb pressures withstood high stresses. Another paper indicated that the moisture condition of the concrete and of the aggregates greatly affects the resistance of the concrete to freezing and thawing. One report was on the nature and amount of changes in volume of concrete while freezing and thawing at various rates. It was brought out in the discussion that the most harmful cycle is one of quick freezing and slow thawing. One of the most interesting reports described the effect of sunlight on the frequency with which a concrete surface goes through a freezing and thawing cycle.

Five reports were presented in the field of reinforced concrete. It was shown that stretching reinforcement bars through the plastic range before embedment increased the elastic and ultimate strength of beams. Results were also reported on pull-out tests on 17 patterns of deformed bars. The strength, deflections and impact resistance of beams reinforced with tempered glass were reported. General test methods, including the use of electric resistance strain gages, for a large scale investigation of reinforced concrete footings was presented. A little different was the report on the study of the maximum positive and negative moments and maximum shears corresponding to floor loading patterns in heavy storage ware-

houses which would give the greatest cumulative effect on a particular beam or girder. Comparisons were made with the maximum moments thus obtained with those determined from the simplified loadings prescribed by the ACI Code. For practical design purposes the results were reassuring.

Speakers included (alphabetically arranged) N. L. Ashton, A. T. Goldbeck, Douglas McHenry, T. D. Mylrea, J. C. Pearson, F. J. Simpson, A.U. Theuer, R.C. Valore, Jr. Stanton Walker, H. E. Wessman, R. R. Zipprodt.

Contributors and questioners from the floor included Leo Coff, Henri Marcus, George Winter, C. M. Wetzel, W. F. Kellermann, N. H. Withey, F. B. Hornibrook and W. H. Wheeler.

Maintenance and Repair of Concrete

Afternoon, February 20

Vice President Walker, as chairman brought the session to order and without further ado introduced Roderick B. Young, the session leader.

Mr. Young worked on his plans for the program for a long time and deserves much credit for the splendid program he developed. He opened the meeting with a short introductory paper which cited many factors contributing to the deterioration of masonry and concrete structures. The six papers following told of experiences and methods used to restore and strengthen such structures.

Two special methods of restoring and strengthening masonry structures

By J. W. Kelly and B. D. Keatts, (published in the ACI JOURNAL for February 1946), this paper was given in brief by Mr. Keatts who illustrated his talk with many stereopticon slides. Structures and foundations that have been damaged by weather, erosion, scour or settlement have been restored and strengthened by pumping cement-base stabilizing material into small interstices and by filling larger spaces with aggregate which is then embedded in the stabilizing material under pressure. Mr. Keatts described the methods used and showed many examples of successfully completed repairs.

Repair of concrete chimneys with minimum boiler operation interference

This paper by W. M. Bassett and Miles N. Clair, presented by Mr. Clair, described with considerable detail three chimney repair jobs in New England. The experiences of the engineers in solving the many problems they had to overcome in completing the repairs with minimum delay are of interest to others who might be confronted with similar work.

Maintenance and repair of portland cement concrete pavements

This paper by A. A. Anderson, was read by L. F. Andrews in Mr. Anderson's absence. It is published in full in this issue of the JOURNAL (p. 477). A. T. Goldbeck, Stanton Walker and Session Leader Young, participated in the discussion.

Maintenance of heavy concrete structures

Mr. Young gave a brief digest of this paper, published in the February 1946 JOURNAL in the absence of the authors, Clay C. Boswell and Albert G. Giesecke. He described repair methods on a dam and contrasted its construction methods with those which have made repairs on a much older, similar structure unnecessary.

Hydraulic structure maintenance using pneumatically placed mortar

R. W. Spencer presented this paper for the author, W. L. Chadwick, giving experiences of the Southern California Edison Co. using pneumatically placed mortar in repairing concrete hydraulic structures high in the Sierra Mountains.

Maintenance and repair of concrete bridges on the Oregon highway system

This paper, by G. S. Paxson (Published ACI JOURNAL, November, 1945)

was presented by L. H. Tuthill in the absence of the author. Mr. Tuthill added a few remarks of his own based on his personal observations of results of the repairs. Mr. Jackson also told of first hand observations he had made on visiting the site of the work. Additional discussion was by M. A. Swayze, A. B. Cohen, R. C. Sandberg and F. R. McMillan.

A. B. Cohen presented a short paper as discussion on the general theme of the session. It described a repair job on a steel bridge at Scranton, Pa. in which a new concrete floor was laid and the entire floor system strengthened by placing spiral reinforcement over the stringers and buckle plate joints.

F. R. McMillan told an interesting story about removal of forms and early exposure of concrete as a factor in disintegration.

Election, Awards, Sealing Compounds, Atomic Bombs, Should Portland Cement Be Dispersed?

Evening, February 20

Election of officers

The report of the Tellers, Jacob Fruchtbaum and Morgan B. Klock, was made by Mr. Fruchtbaum. Vice-President Harrison F. Gonnerman was elected President. Stanton Walker succeeded himself as Vice-President. Robert F. Blanks, formerly Director-at-Large, was elected Vice-President. Roy R. Zipprodt is a new member of the Board elected from the Second District to succeed Myron A. Swayze, after Mr. Swayze's two terms in that office. Henry L. Kennedy was re-elected Director-at-Large—a three-year term. The following Regional Directors were re-elected: Paul W. Norton, First District; Alexander Foster, Jr., Third District; Frank H. Jackson, Fourth District; Charles S. Whitney, Fifth District; Herbert J. Gilkey, Sixth District. At a subsequent meeting the Board of Direction appointed Frank Jackson Director-at-Large to fill the unexpired term (to 1948) of Mr. Blanks who became a Vice-President. H. P. Bigler, Connors Steel Co., Birmingham, Ala., was appointed Director from the Fourth District to succeed Mr. Jackson. By Board action Harvey Whipple was reappointed Secretary-Treasurer.

Following are the five elected members of the 1946 Nominating Committee: Arthur J. Boase, Chairman, Frank E. Richart, Roy N. Young, Lewis H. Tuthill and P. J. Freeman. With them, under the By-Laws, the three latest Past-Presidents will serve: Douglas E. Parsons, Roy W. Crum, and Morton O. Withey.

President Parsons introduced President-elect Gonnerman, Vice-Presidents-elect Walker and Blanks and Director-elect Zipprodt by asking them to stand as he called their names.

Presentation of awards

In a very brief ceremony President Parsons presented Institute awards as follows: Leonard C. Wason medal for "Noteworthy Research" to Bartlett G. Long, Henry J. Kurtz and Thomas A. Sandenaw for the work reported in their 1945 paper "An Instrument and a Technic for Field Determination of the Modulus of Elasticity and Flexural Strength of Concrete (Pavements)"; Leonard C. Wason Medal for the "Most Meritorious Paper" of ACI *Proceedings*, Volume 41, to Clarence Rawhouser for his paper "Cracking and Temperature Control of Mass Concrete".

The first American Concrete Institute Construction Practice Award for "a paper of outstanding merit" on concrete construction practice to Lewis H. Tuthill for his contribution to the January 1945 JOURNAL of the American Concrete Institute, "Concrete Operations in the Concrete Ship Program."

The Henry C. Turner Medal "for notable achievement or service" went to

John Lucian Savage "in recognition of long and distinguished service in the design of hydraulic structures, including some of the world's most notable dams."

Mr. Kurtz accepted Mr. Sandenaw's medal for him in his absence and Mr. Blanks accepted the medal for Mr. Rawhouser. In accepting the Turner Medal on behalf of Mr. Savage who was on his way to Palestine and China as a consultant, Professor Gilkey said:

"This is a dangerous and difficult assignment for a session as crowded as this one because there are so many things one might say about Mr. Savage. Even though this privilege came to me on rather short notice and unexpectedly, I still could easily overtalk on such a subject.

"Mr. Savage, as recipient of the Turner Medal, is the one Mr. Savage officially of interest tonight. The Turner Medal, as I understand it, is not awarded on the basis of service to the Institute or on the basis of the kind of man one happens to be. It is awarded on the basis of outstanding accomplishment in the field of concrete, Institute or no Institute, man or no man, but I would like to talk a little bit about Mr. Savage not only as Turner Medalist but as to his relationship with ACI, rather unique in a number of ways, and also of Mr. Savage the man.

"First, as Turner Medalist: from the time Mr. Savage graduated from the University of Wisconsin, about 1903, most of his professional career has been with the then very young Reclamation Service, now Bureau of Reclamation. He has seen this era of dams come and grow to the fruition of what it now is. He saw the weaknesses in these large structures; most serious among them the cracks that developed from differential cooling strains; he recognized that with Boulder Dam in the offing these were problems of a very serious nature. Here they were about to erect a single structure in which the volume of the concrete exceeded the entire volume that the Bureau of Reclamation had placed in all of its structures up to that time. For such a giant all the unsolved problems of earlier construction would be enormously magnified.

Harrison F. Gonnerman, the Institute's New President

"Unlike smaller men Savage allowed himself to be neither indifferent nor appalled; he simply viewed these problems with the type of questioning concern that begets answers, and answers he got, some of them so revolutionary as to be epoch making. While Boulder Dam and the other dams were not the work of any one man, and the advances we have made in cement these recent years are not the work of any one man, I can say without fear of contradiction by any that know—that Mr. Savage was the spark-plug for much of those two big developments, the improvements in the design and building of dams and the evolution of low heat cement. Concrete still has problems, plenty of them, but Savage got a lot of things started, directed attention to them, and followed through.

"Now in his relationship to the Institute: Mr. Savage, I think, has never attended a convention of the American Concrete Institute. I believe that he has never pre-

sented a paper to the Institute under his name; he has neither headed an Institute committee nor held Institute office. In spite of this record of seeming passivity, however, I recall that in at least one year, he headed the list of those who sponsored new members for the Institute. That I wouldn't say was necessarily altogether Mr. Savage's doing for I suspect the gang in Denver might that year, when his back was turned, have pushed a few prospects his way. Nevertheless, practically every year the name Savage has been fairly high on the list of new "member-getters."

"Mr. Savage has always had the interests of the Institute very greatly at heart. When we look around the Institute; when we search among those who head its committees and who have supplied papers for the JOURNAL; when we look at the recipients of the medals on this occasion we can see the way the man has multiplied himself—these are Savage's boys. They are the men who are doing the work they are doing because of the research and the work which Savage organized and labored so endlessly to promote. It is clear that in his relationship to the Institute, Mr. Savage has played a unique and vital role.

"Now, as a man: Savage is extremely modest and unselfish. I don't think I have ever known a man so much inclined to set himself to the background, so thoroughly and intensely self-effacing even to the point of shyness. There are one or two stories I wish I could tell about Mr. Savage and his modesty, but perhaps it is as well that time does not permit.

"I am just a bit suspicious, however, of the fact that Savage was recalled so abruptly to Palestine, India, Burma and China in connection with dams, at least one of which is to be greater than the greatest yet built. I am suspicious that—if he had anything to say about it—he may have moved the date of departure up a little just to avoid having to stand on this platform. He is that modest and he probably dreaded that much to be in front of you and hear himself talked about.

"There have been other occasions when I have suspected his motives in being far, far away. While during recent years Savage has been awarded various recognitions and medals, among them, the highest that it is within the realm of engineers to give, the John Fritz Medal; I doubt that he has ever been present or even in the country when a major award was made. Apparently a little mystified that men should deem it necessary to accord recognition for the sorts of things he has so enjoyed doing, there is some evidence that his motto, if he had one, might well be, 'if these honors must come let them at least come painlessly.'

"More I think I must not attempt to tell you about Mr. Savage, but he is a man whose attainments, whose personality, whose everything, is indeed of Turner-Medal calibre."

Following the awards, President Parsons turned the meeting over to Mr. Blanks, session leader for the evening.

Address by retiring President

Due to the crowded program Mr. Parsons touched only a few high spots of his address—to be published.

Curing concrete with sealing compounds

This paper, (p. 493) by Robert F. Blanks, H. S. Meissner and Lewis H. Tuthill, presented by Mr. Meissner, supplies data evaluating concrete curing by membrane treatment in terms of equivalent moist curing. It discusses the preferred methods of curing by use of sealing compounds and outlines a specification and acceptance test for their purchase.

Observations of the effect of atomic bomb on reinforced concrete structures—Hiroshima and Nagaski

Presented by the author, Capt. E. H. Praeger (CEC), USNR, this paper described and illustrated with stereopticon slides the effects of the atomic bomb on concrete structures in the two Japanese cities attacked with this powerful weapon. The illustrations which started with examples of destruction near the point of explosion and continued with examples

progressively farther away graphically showed the effects of the bomb at varying distances from the center of the explosion area. Captain Praeger prepared his paper from information gathered as a member of a group of Navy Officers and civilians sent to Japan by the Secretary of the Navy to make a survey of the effects of the atomic bomb.

Should portland cement be dispersed?

T. C. Powers, gave a brief summary of this paper (published in the November 1945 JOURNAL) and added some supplemental comments. Prepared discussion was presented by Henry L. Kennedy and L. R. Forbrich. E. W. Scripture, Jr. and E. C. Shuman participated in the general discussion.

Concrete Farm Silos, New Developments in Concrete Construction Methods

Morning, February 21

Chairman Blanks, in opening the final session of the convention, made a comment which characterizes both the men and the meetings held in Buffalo. "We hear a lot of discussion about the durability of concrete. After the lengthy sessions which you gentlemen have so very patiently sat through in the last two days, I am wondering why we cannot impart to concrete some of the durability we find in concrete engineers."

Before turning the meeting over to Lewis H. Tuthill, session leader for the morning, Mr. Blanks called for the report of Committee 714.

Concrete farm silos

Committee 714 was formed in the latter part of 1937 to develop a recommended practice for the construction of concrete farm silos. After several years work by the committee under the chairmanship of Professor C. A. Hughes of the University of Minnesota and his successor, William W. Gurney, the report, "Recommended Practice for the Construction of Concrete Farm Silos," was published in the January 1944 JOURNAL with revisions in the January 1946 JOURNAL. On a motion by Mr. Gurney, as chairman of Committee 714, the report was approved by the convention subject to ratification by letter ballot. If two-thirds of the members voting approve, the report will become and be published as an ACI Standard.

Concrete at advance bases

Among the many problems which confronted the Navy in its Pacific Island construction, was the lack of customary types of aggregate for concrete. Coral aggregate, as the material at hand, was used by the Seabees in advanced base construction and Commander I. S. Rasmusson, the author, told about it from first hand knowledge based on twenty-one months in the Pacific Area. See p. 541 this JOURNAL.

Prefabricated concrete structures

This paper by A. Amirikian deals with the principles and application of precasting to such specific structures as storehouses, barges, floating caisson gates, drydocks and hangars. The information presented formed the basis of the Navy's wartime concrete shipbuilding program. It is a companion to the paper by Mr. MacLeay on the "Application of Precast Concrete to Floating Structures," which followed on the program.

Application of precast concrete to floating structures

F. R. MacLeay, Chief Engineer of Corbetta Construction Company, presented this paper as a sequel to Mr. Amirikian's paper to describe and explain the developments of construction methods for pouring the thin concrete walls required in concrete barges and landing craft. Well illustrated with stereopticon slides the paper shows some ingenious methods of constructing the thin-walled units required.

H-beam and prepacked method of pier construction

Presented by C. P. Disney, Bridge Engineer of the Canadian National Railways, this paper described and illustrated with slides a method used by the author in building bridge piers in deep water at a fraction of the cost of the caisson method using H-Beam piers and 'Prepakt' concrete. He included a description of a design for a bridge to replace the Quebec Bridge prepared when told German submarines were operating in the St. Lawrence River during World War II and there was the possibility of their torpedoing the bridge piers.

Harnessing the atmosphere for concrete construction

This paper by V. S. Murray was presented with several slides and a short movie in illustration. Pre-stressed prefabricated structural units for small houses were manufactured and erected with applied vacuum used to remove excess water, and to adjust and remove forms and handle finished units. Mr. Murray believes this method can be successfully applied to the construction of small homes to relieve the present housing shortage and also be applied to the construction of concrete bridges from precast structural units, his primary interest.

The discussion, mostly questions directed at Mr. Murray, was by L. R. Forbrich, N. H. Whitney, R. C. Sandberg, K. P. Billner, H. S. Meissner and Session Leader Tuthill.

Observations on a post-war European journey

This was an informal report by M. A. Swayze who was in Germany last summer to make a survey of the cement industry there. He observed that as a whole German cement plants and cements are inferior to what we have in this country and in Great Britain. Their concrete highways, even though made with poor cement, are in beautiful shape after seven years including the extremely heavy war traffic. Slides shown by Mr. Swayze were mostly of general interest showing war damage with a few of special interest to the concrete field.

WHO'S WHO

A. A. Anderson

author of a paper on "Maintenance and Repair of Concrete Pavements" (p. 477) which was presented as part of the Maintenance and Repair Session of the 1946 Convention, has been Manager, Highways and Municipal Bureau, Portland Cement Association, since 1936. He graduated from Kansas State College with the degree B. S. in C. E. in 1914 and re-received his C. E. degree in 1920. Mr. Anderson has been associated with the A.T. & S.F. Ry., Kansas Highways Commission and, since 1924, the Portland Cement Association. During World War I he served as an Ensign in the Navy. He is a member of Sigma Tau, Phi Kappa Phi and an Associate Member of the American Society of Civil Engineers.

Irvin S. Rasmusson

Commander, CEC, U.S.N.R., graduated in civil engineering from Iowa State College in 1925. Following graduation he spent two and one half years in Louisiana on levee construction, seven and one half years in the Panama Canal Zone on surveys and construction, two years on construction of a navigation dam on the Mississippi River, and another two years with the Little Rock Engineer District.

He has been on active duty in the Navy since August 1940, serving at the Philadelphia Navy Yard, Norfolk, Va., and Davisville, R. I., followed by 21 months in the New Hebrides, New Caledonia, New Guinea and the Admiralty Islands, where he became familiar with coral as a concrete aggregate. While in the South Pacific, the 40th Construction Battalion, under the leadership of Commander Rasmusson, was recommended by General MacArthur for battle honors awarded in the name of the President. Since his return from the Pacific he has been on duty at the Bureau of Naval Personnel in Washington, D. C.

Commander Rasmusson's interest in concrete dates back to 1935 when he was

connected with construction of a navigation dam on the Mississippi River by the War Department. His paper, "Concrete at Advance Base," appears on p. 541.

John R. Nichols

a member since 1928, member of the Standards Committee and former Director and Vice President of the Institute, needs no introduction. His paper "Radiant Heating by Reinforced Concrete" appears on p. 513.

R. F. Blanks, H. S. Meissner and Lewis H. Tuthill

collaborated on a paper entitled "Curing Concrete with Sealing Compounds" (p. 493) which was presented at the 1946 Convention. Members since 1932, 1935 and 1926 respectively, and authors of many papers—they need no introduction to Journal readers.

Messrs. Blanks and Meissner also collaborated on a second paper appearing in this issue "The Expansion Test as a Measure of Alkali—Aggregate Reaction" (p. 517).

Honor Roll

1946 and 1947

In the February issue we reported the Honor Roll only to January 24 and there were changes in the week ending the scoring year on January 31, 1946.

Rene Pulido y Morales, Havana, Cuba, top man, finished with a score of 24 Members sponsored and Roy Zipprodt with 10.

F. B. Hornibrook increased his score from 1 to 2 and Walter H. Price joined the roll with a score of 2. J. E. Kalinka and Charles F. Lewis each proposed one member. Wm. B. Cheek, H. A. Babcock, R. W. Hurlburt, F. W. Panhorst and G. S. Paxson received credit for "half a member" each.

Jacob Fruchtbaum and Charles E. Wuerpel are tied for first place with a score of 5 proposed Members each on the new Honor Roll for the year ending January 31, 1947:

Jacob Fruchtbaum	**5**
Charles E. Wuerpel	**5**
James J. Pollard	**3**
Walter H. Price	**2**
K. E. Whitman	**2**
F. J. Beardmore	1
A. J. Boase	1
M. E. Capouch	1
H. M. Hadley	1
Issac Hausman	1
W. M. Honour	1
Wm. R. Kahl	1
H. Walter Leavitt	1
F. R. McMillan	1
Ben Moreell	1
C. C. Oleson	1
J. C. Pearson	1
C. J. Posey	1
Raymond C. Reese	1
F. E. Richart	1
J. W. Tinkler	1
Lewis H. Tuthill	1
I. L. Tyler	1
Stanton Walker	1

The following credits are, in each instance, "50-50" with another member.

Birger Arneberg	H. D. Humphries
J. M. Breen	Frank H. Jackson
Fred Caiola	Douglas McHenry
Miles N. Clair	Eugene Mirabelli
T. F. Collier	George P. Palo
A. C. Eichenlaub	Herman G. Protze, Jr.
H. L. Henson	R. B. Young

New Members

The Board of Direction approved 60 applications for Membership (47 Individual, 6 Corporation, 4 Junior, 3 Student) received in January and February as follows:

Adams, Andrew, State Highway Comm., Bridge Division, Augusta, Me.

Arosemena, Manuel H., Calle 9a No. 8, Panama City, Panama

Ashton, Edward Lowell, 623 College, Iowa City, Ia.

Baker, C. J., 2 West Grove Crescent, Toronto, Ont., Canada

Banks, W. E., 437 E. Magnolia Ave., Auburn, Ala.

Becker, Robert M., 42 North Gate Park, West Newton 65, Mass.

Body, Joseph M., 531 Peeples St., S. W., Atlanta, Ga.

Bradt, H. A., 15 de Mayo 1615 Pte., Monterrey, N. L.

Campus, Fernand, Universide de Liege, 32 Bd de la Constitution, Liege, Belgium

Chaffee, H. S., 220 Scott St., Buffalo 5, N. Y.

Chamberlin, Wilbur H., U. S. Bureau of Reclamation, 19th & Stout Sts., Denver 2, Colo.

Collins, A. Stuart, 232 Delaware Ave., Buffalo 2, N. Y.

Colorado Pre-Mix Concrete Co., 24th & Delgany St., Denver 2, Colo. (Ben Olds)

Commonwealth Portland Cement Company, Ltd., 4 O'Connell St., Sydney, N. S. W., Australia (R. G. Milligan)

Dobinson, James, 240 94th St., Brooklyn, N. Y.

Dotter, Eugene, 119 71st St., Guttenberg, N. J.

Ernst, Walter S., Volunteer Portland Cement Co., Knoxville 7, Tenn.

Fisher, Wilbur H., Wyoming Highway Dept., Capitol Bldg., Cheyenne, Wyo.

Galatis, Anthony Constantine, 50, Syngrou Ave., Athens, Greece

Gill, James F., 2528 Olopua, Honolulu, Hawaii

Goldstein, H., c/o L. Sonneborn Sons, Inc., Hancox Ave., Belleville, N. J.

Griesbach, Walter, The Foundation Co. of Canada Ltd., Guy & Sherbrooke Sts., Montreal, Que., Canada

Harfst, Walter W., 3406 E. 81st St., Seattle, Wash.

Harrell, W. L., 3311 Ozark, Houston 4, Texas

Hawke, Charles E., McNamara Construction Co., Ltd., 42 Industrial St., Toronto 12, Ont., Canada

Hettler, William M., 67 Dorset Dr., Kenmore 17, N. Y.

Hinds, Chester M., 15th Floor, 75 Federal St., Boston 10, Mass.

Hobbs, Merrill H., 1529 W. Falkland Lane, Silver Spring, Md.

Hogentogler C. A., Jr., 18 Oxford St., Chevy Chase 15, Md.

Itriago, Ildefonso, Sur 5 No. 112, Caracas, Venezuela, S. A.

Klieger, Paul, c/o Portland Cement Association, 33 W. Grand Ave., Chicago 10, Ill.

Kretsinger, Donald G., U.S. Bureau of Reclamation, 214 Customhouse, Denver 2, Colo.

Limbach, Wilton G., Southwestern Portland Cement Co., Osborn, Ohio

Loh, C. K., Y. M. C. A., 224 47th St., New York 17, N. Y.

Lorente, M. J., 25 Tavern Rd., Boston 15, Mass.

McCall, H. C., 25 Berkley Place Columbus, Ohio

McCoy, Walter J., Central Laboratory, Lehigh Portland Cement Co., Coplay Route No. 1, Pa.

McKean, H. P., Erik Floor & Associates, 139 N. Clark St., Chicago 2, Ill.

Mayer, Eugene F., Box 143, Lodge Pole, Nebr.

Miller, H. Hershey, 1635 Briggs St., Apt.5, Harrisburg, Pa.

Missouri Rolling Mill Corp., 68 Manchester Ave., St. Louis 10, Mo. (George H. Ackerman)

Moran, Robert P., 17 Dartmouth Rd., West Orange, N J.

Nagel, George E., Permacrete Products Corp., 1839 Wall St., Columbus 7, Ohio

Parmenter, Frank D., 424 Adelaide St., W., Toronto, Ont., Canada

Paul, Claude, Detroit Testing Laboratories, 554 Bagley, Detroit 26, Mich.

Potts, Robert J., 1412 Amicable Bldg., Waco, Texas

Shao-Fan, Chen, Y.M.C.A., Oak Park, Ill.

Smith, J. Morgan, Georgia School of Technology, Atlanta, Ga.

Super Concrete Emulsions Ltd., 1372 E. 15th St., Los Angeles 21, Calif. (George C. Guibert)

Tedesko, Anton, Roberts & Schaefer Co., 307 N. Michigan Ave., Chicago 1, Ill.

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Proposed Manual of Standard Practice for Detailing Reinforced Concrete Structures

ACI Members in good standing and applicants for membership at the time of mailing (in May, we hope) will receive a special Institute publication —the report of ACI Committee 315. The result of several years of intensive work in cooperation with the Concrete Reinforeing Steel Institute, the Manual will be of assistance to the designer, detailer and student. See p.473, this Journal, for the complete announcement including Table of Contents, List of Drawings and List of Tables.

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page

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Asphaltic Oil-Latex Joint-Sealing Compound*

By BRYANT W. POCOCK†

SYNOPSIS

The development of asphaltic oil-latex compounds for use in sealing expansion joints in concrete pavements is discussed. Laboratory tests devised by the Michigan State Highway Department for evaluating these seals are described and results of field installations in Michigan are reported. Tentative Michigan specifications for this type of seal are given.

INTRODUCTION

The historical background of asphalt-rubber compounds for highway purposes was characterized by the development in England and the Netherlands of various mixtures of latex with bituminous substances. These mixtures retained certain properties of the parent materials, notably the elasticity of the rubber and the adhesion, cohesion and ductility of the bitumen.

While these experiments were being conducted in Europe, the design of expansion joints for concrete pavements was undergoing its initial development in this country. American highway officials, recognizing the need for an effective sealing compound for use in excluding moisture and foreign bodies from expansion joints, tried tars, pitches, asphalts and many other substances. For some time, asphalt was held to be the best material available for use as a joint-sealing compound.

Asphalt, however, had certain disadvantages which soon became apparent. For one thing, expansion joint seals made of asphalt tended to flow at higher summer temperatures. Conversely, they became brittle in winter and lost most of their ductility, cohesion and adhesion to concrete.

It was only natural that American highway engineers should become interested in the progress of European experimentation with rubber and

*Submitted to the Institute, June 18, 1945.
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asphalt mixtures in the hope of devising a joint-sealing compound of superior quality. As early as 1935, the state of California used a material consisting essentially of about 70 per cent SC-4 asphaltic oil and 30 per cent rubber latex as a seal on about 680 linear feet of expansion joints on the heavily traveled highway between Lebec and Grapevine in Kern County (1)*. These joints, spaced at 100-foot intervals, were sealed immediately after the concrete surface was completed. When inspected in 1939, after 4 years of service, the rubber compound joints were intact and in good condition.

Stanton (2) reported early experiences (1936) of the California Division of Highways with the new material, indicating the necessity of choosing a good asphaltic oil and basing its selection on high ductility at low temperatures. Stanton pointed out the deleterious effects of moisture in securing a good bond and showed how to convert the sealing compound into a crack filler by cutting the oil with gasoline before adding the latex.

By March, 1934, Woolf and Runner (3) had set up a procedure for testing various types of "resilient expansion joint fillers" in the laboratory. Among the nine types of fillers investigated were compounds of asphalt and rubber with particles of vulcanized rubber.

Encouraged by the results in California, the neighboring state of Oregon experimented with a seal made up of 30 parts of 60 per cent rubber latex and 70 parts of 150-200 penetration asphalt. The mixture was poured hot and the seal allowed to come within ¼ inch of the pavement level. The hot material was covered with rubber grindings to complete the seal and filling of the joint.

Impressed by the work in California and Oregon, engineers in Massachusetts and Oklahoma began experimenting with similar compounds. A formula was developed in Oklahoma for a joint seal using the following mixture:

Material	Parts by weight
SC-4	5.475
38 per cent Latex	2.431
Lime	0.0553
Cresol Soap Solution	0.0444
Paraffin	0.0395

The soap solution used in the formula was a proprietary material of the brand name "Kremulso." Experience in Oklahoma prior to 1939 (4) indicated that good joint seals are readily obtained with the above mix if the following precautions are taken: 1) thorough cleaning of the concrete sides of the joint; 2) accurate proportioning of materials; 3) uni-

*The numbers in parentheses refer to the bibliography appended hereto.

formity of control of mixing, temperature and consistency of each batch; 4) pouring of a uniform flush joint (seal); 5) delay of shouldering operations for one week after pouring of joints and 6) the exercise of care that joints are not poured at temperatures below 40 F.

As progress in the development of bitumen-rubber materials in the United States came to the attention of highway engineers, various proprietary or commercial preparations appeared on the American market, some of which showed considerable promise, and a few of which gave good results in actual service. Nevertheless, research continued in highway departments and other laboratories (5-9) on problems associated with the development of latex-bituminous types of joint-sealing compounds having superior properties.

The Michigan State Highway Department concerned itself at an early date with these investigations and undertook the development of a bituminous-rubber sealing compound suitable for use on expansion joints in Michigan climates (10). In December, 1936, project 36 G-4 was authorized by the Administration for the investigation and development of new joint sealers, notably types using normal latex and road oil. Whereas considerable success has attended this project during its eight-year life, the investigation is by no means closed. At the present time consideration is being given to the use of synthetic latices as substitutes for normal latex.

DEVELOPMENT OF COMPOUNDS IN MICHIGAN

Michigan investigations of joint-sealing compounds were first patterned very closely after those of California insofar as materials and properties were concerned. However, the scope of the project was soon enlarged to take into consideration many significant factors relative to the behavior of these compounds, including properties, materials, admixtures, preparation and handling of the compounds in the field.

It was early determined that a mixture of latex, lime and road oil of the SC-4 type A.A.I. (1940 designation) gave the best results. In this mix prevulcanized, specially stabilized latex of 58.8 per cent concentration of solids produced a very tough, elastic sealer, which set up rapidly. The workability of this mix was improved by adding paraffin and varying the percentage of lime. Paraffin also increased the toughness of the set mix, but it tended to lower adhesion and increase "graininess." Its use, therefore, was discontinued. Little success was obtained with powdered rubbers, probably because of their relatively larger particle size and the consequent slowness with which these materials disperse in road oil. Adhesion and ductility were gravely impaired by the addition of such inert fillers as bentonite (colloidal aluminum silicate), fine sawdust, diatomaceous silica, Fuller's earth and cotton

in percentages sufficiently high to appreciably promote toughness and, therefore, were not given further consideration.

The sealing compound has been found to undergo a steady, prolonged vulcanization over a period of years, resulting in the progressive hardening observed in mixes several months after pouring. Attempts to control vulcanization by developing a vulcanizing agent of optimum activity included investigations using sulfur flowers and sulfur monochloride. Results indicated organic sulfur to be more satisfactory as a vulcanizing agent than inorganic sulfur. The problem was complicated by the necessity of maintaining suitable workability for pouring.

According to the work of H. L. Fisher (11), all vulcanizing agents are either oxidizing agents or require the presence of oxidizing agents. If this view is accepted, the presence of sulfur in the oxidizing group is not necessary, for the fundamental reaction is one of oxidation-reduction. There are many substances besides sulfur capable of giving rise to such a reaction. Several were tried out, such as piperidine pentamethylene dithiocarbamate, 50 per cent aqueous sodium mercaptobenzothiazole, zinc oxide and casein.

In general, it was found that although the use of these vulcanizing agents produced laboratory mixes whose properties when set were highly desirable, the quantity of vulcanizing material required was so critical that a variation of about 1 per cent would determine the difference between excellent workability and no workability.

The relative failure of sponge rubber to extrude under pressure, its comparative toughness when well vulcanized, and the economic feasibility of using 50 per cent air in the mix prompted an investigation of the possibility of producing voids in the sealing compound, and of the influence of such voids upon its properties.

The method used employed the chemical reaction which occurs between sodium hydroxide and powdered aluminum, during which molecular hydrogen gas is liberated:

$$2\,Al + 6\,NaOH \rightarrow 2\,Na_3AlO_3 + 3\,H_2$$

It was found that a mix with delayed volume expansion and satisfactory workability could be produced, but that here again the quantities of materials used were too critical for operations in the field. The properties of the resulting compound were too susceptible to very slight alterations in the technique of manufacture.

A satisfactory joint-sealing compound was finally developed in which prevulcanized latex was substituted for normal latex. This material adhered satisfactorily to concrete, remained ductile and displayed low susceptibility over a wide range of temperatures. It possessed satisfactory workability for pouring. Specifications for this seal are given at the end of the text.

The mixing and application of the compounded joint material in the field presented a trying problem. Various mechanical devices were tried with the object of mixing and pouring the compound in one or two operations. Any such method was found unsatisfactory, because the rate of set could not be effectively controlled. Finally, it was found that the asphaltic oil and latex could be satisfactorily combined in small batches (3 gal.) by either mixing by hand or by a small mechanical agitator. After proper blending of the materials the compound could then be readily transferred to a standard pouring pot for subsequent filling of the joint opening.

FIELD PROJECTS SEALED WITH ASPHALT-LATEX COMPOUND

By 1940, sufficient experimental work had been done by the Department to warrant the use of the asphalt-latex sealing compound on regular concrete pavement projects. The task of mixing the compound and sealing the joints became the responsibility of the contractor and his personnel. Expansion joints containing one-inch bituminous premolded fiber boards were sealed on two complete projects with asphalt-latex joint-sealing compound before the supply of rubber latex was stopped by the War Production Board. These projects include the design section of the Michigan Test Road, projects 18-20 C3 and 67-37 C4 on M-115 between US 10 and M 66 northwest of Farwell, Michigan, and the Grand Rapids-East Belt construction project F41-34 C6 on US 131 bypass south of US 16.

Michigan Test Road

The joints on the Michigan Test Road were sealed in the summer and fall of 1940, using sealing compounds made with both normal latex and prevulcanized latex for comparative study. The mixing equipment and method of pouring the joints are illustrated in Fig. 1a and 1b. Preparation of the joints prior to pouring was done in accordance with accepted practice for any type of joint-sealing material. In general, the operation was done in the following manner:

The asphaltic oil was heated to approximately 90 C. in tar kettle (A), Fig. 1a. To mix a batch of asphalt-latex material, a quantity of hot asphaltic oil was drained into mixing drum (B) and lime added and mixed into the asphaltic oil. The correct amount of latex was allowed to flow slowly from container (C) into container (B) while constantly stirring to thoroughly blend the ingredients. The finished mixture was then transferred from (B) into pouring can for subsequent sealing of the joints as illustrated in Fig. 1b.

Compounds made with both normal and prevulcanized latex have given excellent service during the first five years without any maintenance whatsoever. However, present conditions indicate that the

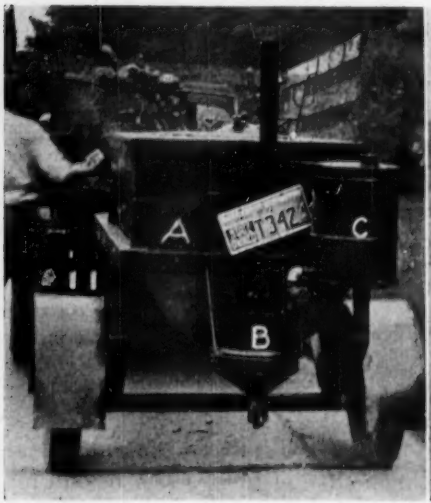

Fig. 1a. (left)—Equipment used in preparing asphalt-latex joint sealing compound.
A—Standard tar kettle; B—5 gal. mixing drum; C—5 gal. can to hold latex.

Fig. 1b. (right)—Pouring asphalt-latex joint sealing compound. (Note rubbery consistency of the material.)

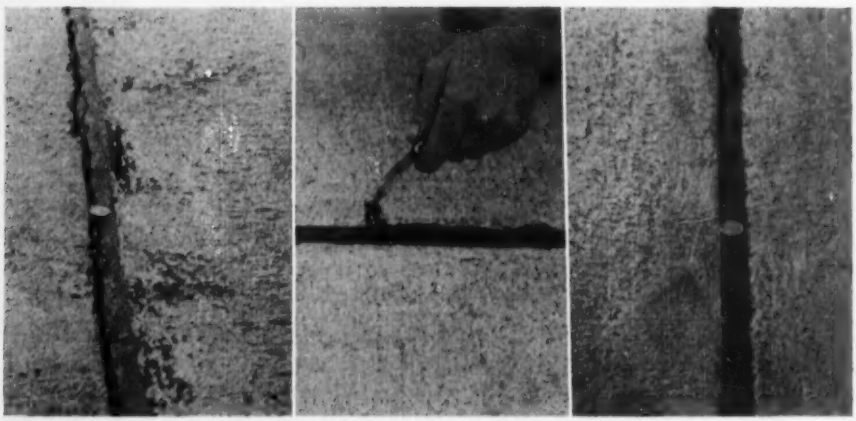

Fig. 2a. (left)—Joint seal containing normal latex after 4½ years in service. Note partial failure in cohesion which appeared after 3 years in service. This phenomenon takes place in cold weather and disappears during summer months. Photograph taken March 21, 1945.

Fig. 2b. (center)—Joint seal material containing normal latex after 4½ years in service. Note consistency of material during warm weather. Photograph taken August 15, 1944.

Fig. 2c. (right)—Joint seal containing pre-vulcanized latex. Note that this material does not fail in cohesion during cold weather and possesses better durability qualities than those of the joint seal with unvulcanized latex. Photograph taken March 21, 1945.

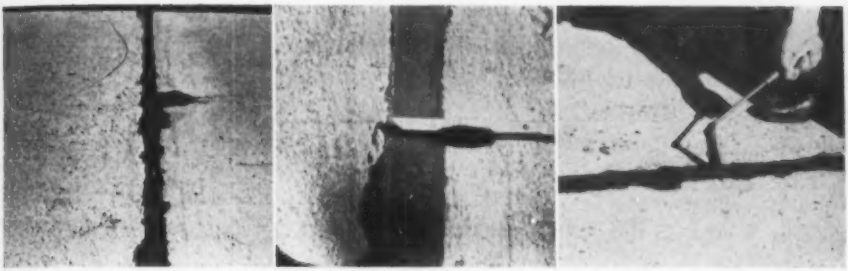

Fig. 3a. (left)—Joint seal containing pre-vulcanized latex. Joint sealed October, 1941. Photograph taken October 19, 1944.

Fig. 3b. (center)—Joint seal containing pre-vulcanized latex. Note unusual width of joint and perfect bond of joint seal material to concrete. Same joint as illustrated in "a". Photograph taken October 19, 1944.

Fig. 3c. (right)—Plasticity of the joint seal material illustrated in "a" after 3 years in service. Temperature 50 F. Photograph taken October 19, 1944.

prevulcanized latex produces a more durable compound. Fig. 2a, 2b and 2c illustrate the general condition of the material in the spring of 1944 and 1945. The compounds have retained their excellent bonding and plastic characteristics. It has been observed that at some points the joint seal has cracked or has lost some of its cohesive properties during the winter season although the material has remained plastic. In the summer these cracks disappear and the material appears to have considerable life. This phenomenon is more common in the compound made with normal latex and evidently this is the first stage in the ultimate failure of the joint-sealing compound. It may be possible to correct this weakness where it first appears by an application of some appropriate treatment. This is being studied.

Grand Rapids—East Belt

This project was constructed the following year, 1941, by a different contractor, but the joints were sealed in the manner described above, except that prevulcanized latex was used in preparing the seals. The general condition of the joints after 3 years is illustrated in Fig. 3a, 3b and 3c.

Quantity and cost per joint

In the case of a normal expansion joint of 1-inch opening, the concrete slab being 22 feet wide and the depth of sealer being $\frac{3}{4}$ inch, the total volume of sealer used in each joint would be 198 cu. in. This would be equivalent to 0.857 gal. To allow for waste, a quantity of 1 gal. of compound per joint seems to be a reasonable amount for estimate purposes.

Based on 1940 prices of materials in large quantities (50-gal. drums for road oil, 10-gal. drums for latex and 50-lb. lots for lime), a specific

gravity of 0.981 for road oil, a specific gravity of 0.949 for prevulcanized latex, and a specific gravity of 0.97 for the final product, the following material costs can be stated for the oil-latex joint sealer:

(a) Per lb $0.0842
(b) Per gal $0.68
(c) Per joint $0.68

SEALING OF JOINTS WITH ASPHALT-LATEX COMPOUND

Experience in Michigan has shown that asphalt-latex joint-sealing compounds made and applied in accordance with the following practices are the most satisfactory.

Preparation of the joint

As stated in the 1942 specifications of the Michigan State Highway Department, the tops of expansion joints and all edged joints must be sealed as soon as the curing agent is removed and before any traffic is permitted on the pavement. Joint openings must be thoroughly cleaned, all contact faces wire brushed, and surfaces must be dry when the seal is poured. Any membrane curing material remaining after wire brushing would be almost certain to afford a good bond for the asphalt-latex compound.

Preparation of the asphalt-latex compound

A tank of at least 25 gal. capacity must be provided for heating the oil (Fig. 1a). The mixing tank should be of 5 to 10 gal. and a stirring device must be used which will produce a homogeneous mixture of uniform consistency.

The following steps are followed in compounding:

a. Heat oil to temperature of between 85 and 95 C.
b. Measure correct quantity of lime.
c. Measure correct quantity of oil.
d. Measure correct quantity of latex.
e. Pour correct quantity of hot oil into mixing tank.
f. Add correct quantity of lime to oil in mixing tank.
g. Stir lime into oil rapidly until thoroughly dispersed.
h. Continue to stir while adding approximately one-half of the correct quantity of latex to the mixing tank. Continue to stir until thoroughly blended.
i. Continue stirring while adding balance of latex until mix is homogeneous and of desirable thickness.

No more compound should be made than can be poured before it becomes too thick for handling. Stirring can be done by hand or mechanically. A 1-gal. container with a handle on one side and a spout on the other has been found suitable for pouring. In general, it may be

said that 5 gal. of compound may be prepared at one time and still remain sufficiently workable for the operator to pour the entire amount. Compound or oil can be removed from equipment with kerosene or other organic solvent. Traces of latex can be flushed out with water when wet or removed by mechanical stripping when dry.

LABORATORY TESTS

Certain tentative laboratory tests have been developed by the Michigan State Highway Department for evaluating and comparing various properties of joint seals (10). The properties studied for which tests were devised included ductility (cohesion), adhesion and flow properties (viscosity).

Ductility and adhesion

A standard asphalt ductility machine was employed, using specially constructed steel forms. (Fig. 4a). These forms functioned to grip regular mortar briquettes which were molded with a brass strip imbedded through their centers in such a way that they left the molds in symmetrical halves, each half presenting a working surface of one square inch cross section.

Preparation of the Briquettes. A cement mortar consisting of two parts natural sand to one part portland cement with sufficient water for a troweling consistency is prepared. This mixture is placed in gang molds of the type used in making A.S.T.M. tensile strength briquettes. A metal brass divider is placed in the center of the mold so as to divide the briquette into two symmetrical halves of one square inch working

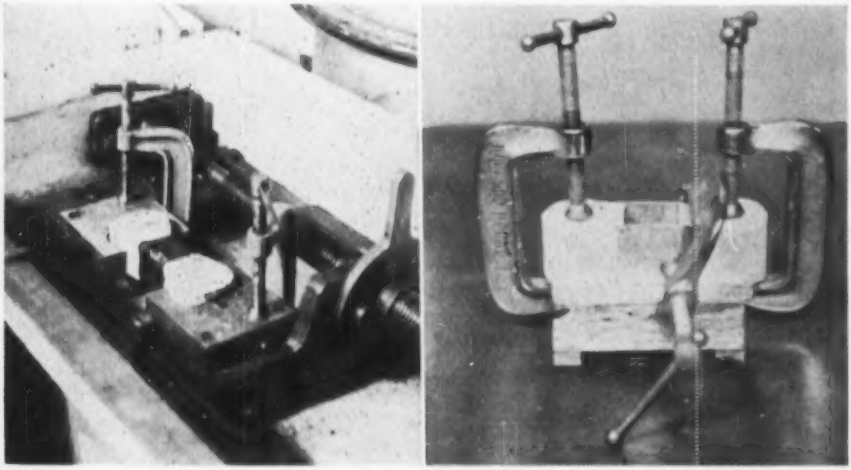

Fig. 4.—Ductility and Adhesion Test: a. (left)—Specimen in place ready for test; b. (right)—Mold assembled prior to pouring in joint seal.

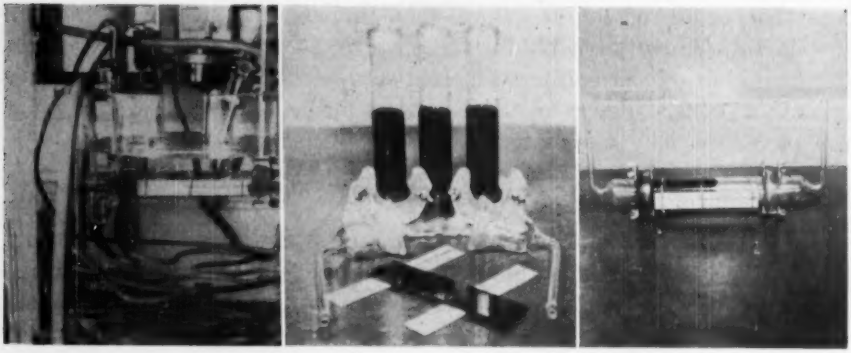

Fig. 5a. (left)—Viscometer used to study flow properties of joint sealer.
Fig. 5b. (center)—Flow cells used with viscometer.
Fig. 5c. (right)—Flow cell assembled and ready for determining viscosity of joint seal.

surface area each (Fig. 4b). Specimens not having perpendicular faces are rejected.

The briquettes are cured in the moist room for one week, then allowed to air dry before use. The faces are ground on a glass plate prior to use with Grade F Carborundum powder to remove irregularities and laitance material after which they are washed and surface dried.

Preparation of the Sealer. A mercury amalgamated brass mold consisting of two side pieces, bottom plate, and three "C" clamps is attached to opposite briquette halves so that a 1 cu. in. space between the faces can be filled with the sealing compound to be tested. (Fig. 4b). The compound is heated to its application temperature, stirred and poured into the 1 cu. in. space a little higher than the upper surface of the briquette.

Method of Conducting the Test. After the sealing compound has been poured, the specimens are allowed to stand over night and excess compound over 1 cu. in. is trimmed off with a hot knife. The clamps are removed and the side pieces and bottom plate are disassembled. The specimens are placed in a refrigerator to prevent deformation.

A standard ductility machine is adapted for operation at the rate of 1 in. per hr. by use of necessary speed reductions. The glycerine bath used for low-temperature measurements is maintained at 0 F. by suitable additions of crushed dry ice and adequate stirring. Specimens are placed in the bath a half hour before testing. Fig. 4a shows the specimen in place and ready to be tested.

The distance the specimen can be drawn in inches before failure in bond (adhesion) or in ductility (cohesion) is recorded, and the type of failure is noted.

Viscosity

The constant temperature bath of a Kopper's viscometer was used in developing a test for the rheological properties of joint-sealing compounds, as shown in Fig. 5a. Special flow cells were designed (Fig. 5b) embodying the use of semi-ball ground-glass joints and clamps, which were furnished by the Scientific Glass Company. The test was based on a method proposed by Bingham and Stephens (14), making use of the formula of Pitman and Troxler (15):

$$\eta = \frac{PR^2t}{4\,h\,L}$$

where η = viscosity in poises
P = pressure applied in dynes per cm.^{2}
L = length of column of seal, cm.
t = time of flow, sec.
h = length of extrusion in cm. in time t
R = radius of tube in cm.

Viscosities were plotted on the standard A.S.T.M. chart, on which the double log of viscosity versus the log of the absolute temperature is a straight line function. The slope of this line is called the susceptibility of the material, and represents the degree to which its viscosity is affected by temperature. This value is also referred to in the literature as viscosity index.

Pouring of Sealing Compound into Flow Cell. The compound is heated to application temperature, melted, mixed to homogeneity and poured into the flow cell (Fig. 5b). The specimen is allowed to stand over night.

Assembly of Flow Cell. The cylinder is removed from the amalgamated plate, the unit is assembled in a horizontal position as in Fig. 5c, and special clamps are placed in position to hold the glass joints in place. Millimeter scales are placed at the rear and front of the tube.

Test Procedure. The flow cell unit and scales are placed in the constant temperature bath and left for 30 minutes. The vacuum hose is attached to one end of the cell, the other end being left open to the atmosphere unless more pressure is required to obtain flow. The time of flow from the ½ cm. mark to the 1 cm. mark is recorded, as well as the pressure indicated on the manometer.

Practical application of laboratory tests

The foregoing laboratory tests were conducted on joint-sealing materials of the following types:

Types 1 and 6. Proprietary brands of asphalt-rubber joint-sealing compound of the hot-pour type.

Type 2. Commercial joint seal of the asphalt type.

Type 3. Experimental asphalt-rubber compound not commercially available.

Type 4. Seal made of 30 per cent prevulcanized latex and 70 per cent SC-6 asphaltic oil.

Type 5. Seal made of 30 per cent normal latex and 70 per cent SC-6 asphaltic oil.

The above seals were so made as to be free of extraneous water, coal tar and other tar products, pressure still residua, pitches, or other products of decomposition. Manufacturing specifications for type 2 were such as to preclude heating the bituminous material above 700 F. or exposing it to pressures appreciably above atmospheric, and to assure that the finished product possess characteristics within the following limits:

1. Specific gravity at 77 F.	1.22 to 1.30
2. Flash point, Cleveland open cup	Not less than 350 F.
3. Melting point, ring and ball	122 F to 135 F.
4. Penetration at 77 F., 100 gm., 5 sec.	40 to 50
5. Ductility at 77 F.	Not less than 30 cm.
6. Loss at 325 F., 50 gm., 5 hrs.	Not more than 3.0 per cent
(a) Penetration at 77 F., 100 gm., 5 sec.	Not less than 20
7. (a) Bitumen sol. in CS_2	62 to 75 per cent
(b) Ash on ignition	20 to 32 per cent
(c) Difference between 100 per cent and sum of (a) + (b)	Not less than 4 per cent

Results of the adhesion and cohesion (ductility) studies on the above materials are shown graphically in Fig. 6. All specimens were drawn in the ductility bath at a rate of 1 in. per hr. At this low rate of drawing, all specimens gave good results at room temperature. As a matter of interest and for comparison a 24 penetration asphalt was included in this group of tests. This material failed at a distance of 0.125 in. In Fig. 6, distance drawn is plotted against temperature of the bath, the lowest temperature being 0 F. At 0 F., seals No. 4 and 5 went to the limit of the test (2 in.), whereas seal No. 1 went 0.625 in. before failure and both No. 2 and No. 3 failed in adhesion and cohesion at 0.125 in.

In carrying out adhesion and cohesion studies, it was found virtually impossible to get good adhesion to the mortar briquettes when the latter were cold and damp. This fact suggests the significance of cold, damp weather during joint sealing operations and the practical importance of developing a means of eliminating or minimizing its deleterious effect on adhesion of joint-sealing materials to the slab. No such means has yet

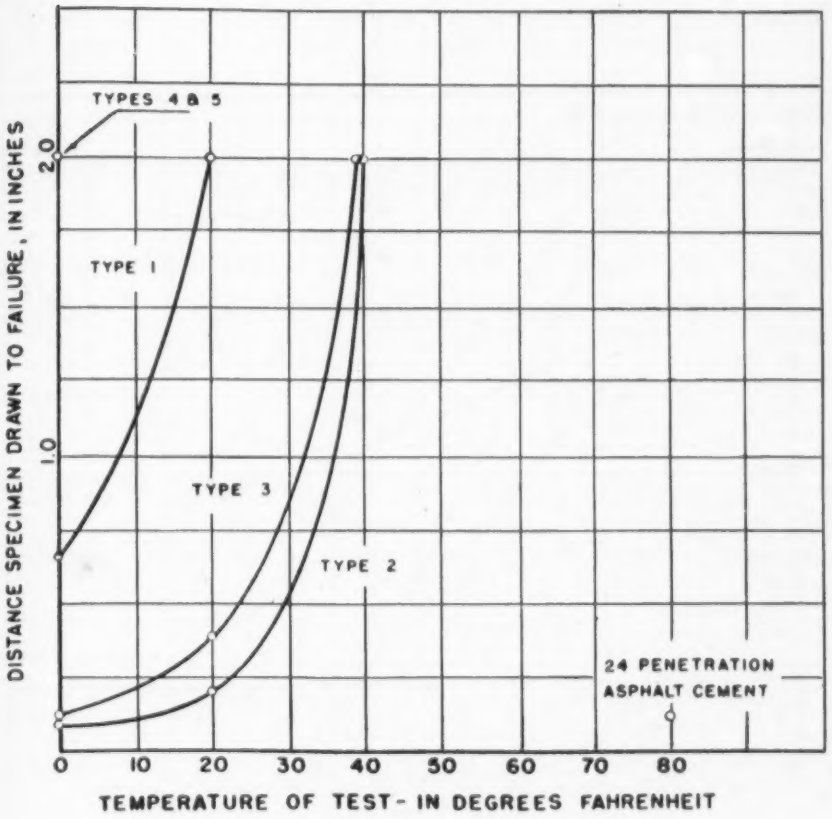

Fig. 6.—Effect of temperature on adhesion and cohesion.

been found satisfactory other than straightforward cleaning and drying of the joint surfaces.

Flow rate properties of the five types of joint seal as measured by the apparatus described above are plotted in Fig. 7.

CURRENT SPECIFICATIONS FOR ASPHALT-LATEX JOINT SEAL

The following additional specifications are given by the Michigan State Highway Department:

The vulcanized latex shall be an aqueous dispersion of rubber particles in an ammoniacal solution, having a rubber solids content not less than 58 per cent. It shall be kept in sealed containers to prevent evaporation of the emulsifying agent. The vulcanized latex shall be protected from temperatures less than 40 F. and greater than 100 F.

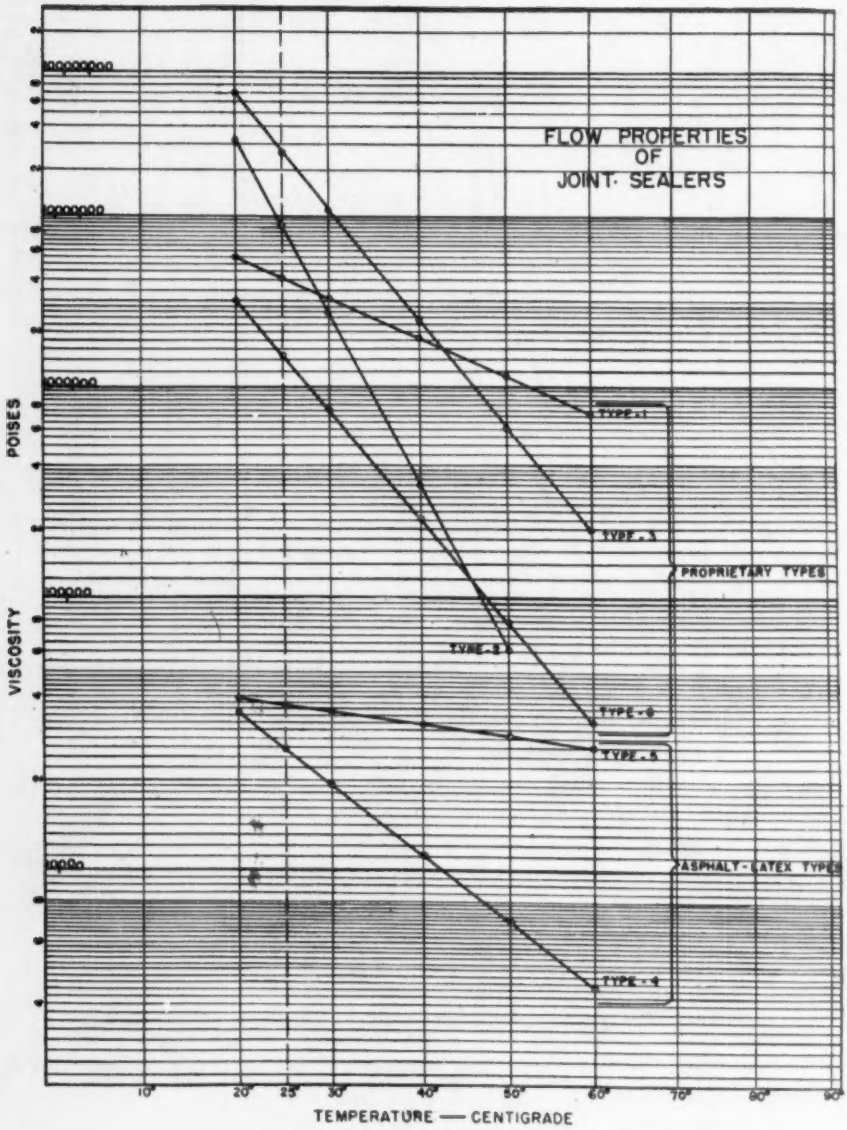

Fig. 7.—Flow properties of joint sealers.

The asphaltic oil shall be homogeneous, shall not foam when heated to the required temperature and shall conform to the following requirements:

	Minimum	Maximum
Specific gravity, 25/25 C.		1.0
Water, per cent by volume		0.5
Flash point, Cleveland open cup, deg. C.	120	
Loss on heating at 163 C., 50 gm., 5 hr., per cent		6
Viscosity, Saybolt Furol, at 60 C.	350	600
Solubility in carbon tetrachloride, per cent	99.5	
Oliensis spot test	Neg.	
Residue of 100 penetration:		
Per cent residue	70	
Ductility, 25 C., cm.	100	
Ductility, 4 C., cm.	7	
Sulfur content, per cent	2	4

Hydrated lime shall conform to the requirements for Mason's hydrated lime in the 1942 specifications for Hydrated Lime for Structural Purposes, A.S.T.M. Designation C 6.

The proportions by weight of the mixture shall be:

Lime, $Ca(OH)_2$	2 parts
Vulcanized Latex	30 parts
Asphaltic Oil	70 parts

Compounds made in accordance with the above procedures have given satisfactory performance in Michigan over a period of 4 years.

ACKNOWLEDGMENTS

The writer is indebted to Messrs. George A. Mansfield and Thaddeus Wolczynski, formerly of the Michigan State Highway Research Laboratory staff, for their early researches on important fundamental and practical aspects of asphalt-latex joint-sealing compounds.

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Vol. 18 No. 4 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN Part 2 Dec. 1946

Discussion of a paper by Bryant W. Pocock:

Asphaltic Oil-Latex Joint-Sealing Compound*

By THOS. E. STANTON† and AUTHOR

By THOS. E. STANTON

Supplementing the excellent paper by Bryant W. Pocock on the development of asphaltic oil-latex compounds for use in sealing expansion joints in concrete pavements, it is in order to discuss the long time performance of the first material of this type which was developed and used by the California Division of Highways in 1936 and the modifications in the original product which have been made since the first report on the California work which was published in "California Highways and Public Works," September, 1936. A recent inspection of the original joints shows the material to be still in excellent condition after ten years with no maintenance or renewal during the decade (Fig. Aa).

Although this original material (designated California Type I) appears equal, if not superior to any subsequently developed product, there are certain difficulties in compounding and handling which led to studies looking to the development of a product which could be more readily handled and which could, if possible, be prepared at some central location, kept in storage indefinitely without deterioration and then shipped to the field for use without heating.

We have been successful in developing such a product (designated as California Type II) the first installation of which was made about six years ago and which is still in excellent condition. (Fig. Ab) Indications are that this material has a service life of well over ten years. Although possessing some deficiencies, hereinafter described, this Type II material is now used as standard in preference to the original Type I, primarily because of ease in handling.

No particular economy in cost of ingredients or marked superiority in performance is claimed for the California material over several ex-

*ACI JOURNAL, June 1946; *Proceedings* V. 42, p. 565
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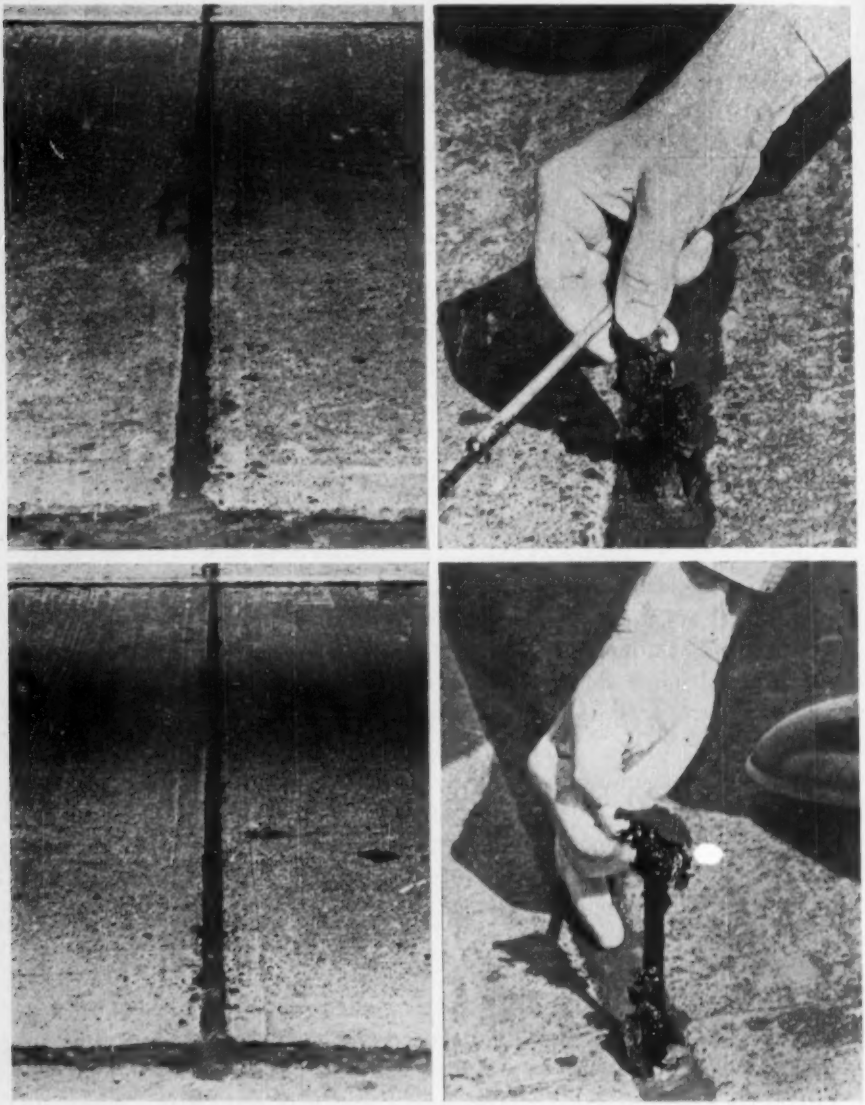

Fig. A—Showing long time performance of two California Type asphaltic oil-rubber latex joint fillers.

a. (top) California Type I poured in 1936. Condition after ten years.
b. (bottom) California Type II poured in 1940. Condition after six years.

cellent commercial products. However, all known commercial products require special equipment for heating and pouring, whereas the California product made in accordance with the specifications embodied in this discussion does not require any heating. At the same time, because of its fluidity, greater care must be exercised to insure against leakage from the filled joint space while still in a fluid condition making it is necessary to return later for a second filling to bring to grade.

In general the procedure is to emulsify a heavy road oil (SC-4,5 or 6) with rubber latex (natural or synthetic) in the presence of an emulsifying agent (sodium silicate). Such a material properly compounded can be kept several months without deterioration if stored in sealed cans. It is shipped to the field in this condition and just before pouring into the crack or joint a small amount (3 to 5 percent by weight) of powdered sodium silico fluoride (sodium fluosilicate) is added by sifting the powder into the liquid and stirring until thoroughly mixed. The sodium fluosilicate induces a gelling of the compound and solidifies it into a resilient mass which stiffens progressively as the water in the emulsion is eliminated. There is an excellent adherence to the concrete.

As any asphalt-rubber type crack filler is expensive because of the high cost of rubber, studies are being conducted using rice hulls as an extender. Several joints of this type using 50 percent by volume of rice hulls, poured eight months ago, are still in sufficiently good condition to warrant further trial.

The following instructions have been prepared for guidance in the manufacture and pouring of the California Type II seal:

INSTRUCTIONS FOR PREPARING AND MIXING ASPHALT-LATEX JOINT SEAL CALIFORNIA TYPE II

California Type II asphalt-latex joint seal is prepared by emulsifying a heavy road oil or liquid asphalt, such as an SC-4, 5 or 6 grade, with commercial rubber latex in the presence of an emulsifying agent (sodium silicate). The mix may be used immediately or, if properly made, may be stored for several months in sealed cans without deterioration.

The manufacturing equipment, materials and procedure are as follows:

Equipment

a) Two 5-gal. pails, b) two thermometers, 0-220 F, c) wooden paddles for stirring, d) some means for heating the oil and rubber latex and maintaining a controlled temperature. (The materials may be heated either in the drums or in special tanks depending upon quantities to be handled) e) a mixing vessel (a 50-gal. drum cut in half and equipped with a wide vane rotating paddle makes a convenient mixing pot.)

Materials

a) Commercial rubber latex (39-40 percent solids). The latex may be either plantation rubber or Buna-S synthetic latex of the variety designated as GRS-Type 3.

b) Liquid asphalt grade SC-4, SC-5 or SC-6. (All asphalts are not equally suitable for this purpose and it has been found that asphalts from some sources emulsify much better than others.)

c) Sodium silicate commercial grade. (Diluted with an equal volume of water before using.)

Mixing

a) Heat 40 parts by volume of rubber latex combined with from 5 to 10 parts by volume of sodium silicate (½ strength) to a temperature of between 110 F and 140 F.

b) Heat 60 parts by volume of liquid asphalt to a temperature of between 180 F and 205 F and add *slowly* to the rubber latex stirring steadily as the asphalt is added. (It is particularly important to introduce the asphalt slowly at first until the emulsion has started to form after which the asphalt may be added more rapidly. Relatively slow gentle stirring, such as may be readily accomplished by hand rotation of the paddle, has seemed to be more effective than rapid agitation with a power driven impeller. When all of the asphalt is added, the mixture should have the consistency of ordinary house paint and the best emulsions will have a smooth uniform texture and a brownish color. Bluish-gray or black colored mixtures are usually not as satisfactory, are less stable and tend to separate or coagulate when stirred).

c) Pour the mix into cans or drums equipped with tight fitting lids in order to protect from the air. These mixtures have been kept in storage for a number of months with no deterioration although at times it is helpful to mix a little water with the seal if it should become too stiff.

When ready for pouring into a joint, a small quantity (3 to 5 percent) of sodium silico fluoride is rapidly mixed with the asphalt latex combination which is then poured immediately in place. Sodium silico fluoride (in the form of a white powder) has a gelling effect on rubber compounds and will promote early setting. The consistency may be adjusted somewhat by the amount of powder used.

NOTE: Like all emulsifying processes, the asphalt rubber latex mixtures show occasional eccentricities or departures during the mixing or manufacturing process and in spite of ordinary care, occasional batches have failed to work out as intended. Thus far in California experience, it has been possible to keep these occasional failures to a small percentage of the total; nevertheless, minute percentages of contaminates in the asphalt or latex may have a marked effect upon the quality of the mixture.

INSTRUCTIONS FOR POURING CALIFORNIA TYPE II ASPHALT-LATEX JOINT SEAL

1) Prepare joints for sealing by removing all dirt, loose concrete, etc. from the joints. (Be sure that the ends of the joints have been properly cleaned to the full depth.)

2) In case the shoulder material is not in place, plug the end of the joint with a temporary earth dam or a small piece of rag.

3) Open the can of Asphalt-Latex Seal, skim off surface skin, if any, and inspect sealing compound for uniformity and consistency.

4) If any separation or settling has taken place, a little stirring should produce a uniform consistency. If too thick, the can should be warmed or a little water added to thin to the proper consistency. (The seal should be about the consistency of house paint to pour readily.)

5) Add a small amount (3 to 5 percent by weight) of powdered sodium silico fluoride (sodium fluosilicate) by sifting the powder into the liquid and stirring until thoroughly mixed. The proper amount to use will have to be determined by trial so that the seal

will harden or set in about 30 to 60 min. The more powder added, the faster the seal will harden. Adding water will slow down the setting time of the seal and additional powder will have to be used. (It is more convenient to measure than weigh the powder.) About 4 to 7 tins (3 oz. gill type) of powder will usually be required for five gal. of seal.

6) Mix only as much seal and powder as can be poured in about 10 min. After the seal has once hardened it cannot be resoftened.

7) Keep the seal in closed can when not in use, as it skins over rapidly when exposed to sunlight and air.

8) Pour the seal (after mixing with the powder) from small cans or pouring cornucopias with a wide mouth.

9) The seal can be washed off the cans or cornucopias with water before it startes to harden. After it has hardened it can be burned off with a kerosene torch. Kerosene or gasoline can be used to clean cans or tools after a little soaking to soften the seal.

10) Dust the surface of the seal in the joints with cement or very fine dust to prevent its being picked up by traffic before it has hardened.

11) The joints should be filled with seal flush with the surface in warm weather. The seal shrinks a little on hardening and it is usually necessary to refill the joints the following day.

NOTE: The sodium silico fluoride powder is a poison and care should be taken not to get any into the mouth or lungs. The hands should be washed thoroughly after handling.

AUTHOR'S CLOSURE

With reference to Mr. Stanton's comprehensive report on asphaltic oil-latex joint-sealing compounds developed in California, in particular the product designated as California Type II, it would seem appropriate to record a few relevant observations made in this laboratory* and not included in the author's original paper.

The many advantages associated with a sealing compound made up months in advance of its use and requiring only the simplest kind of field manipulation for actual sealing have been appreciated for some time in Michigan. These advantages have been set forth in Stanton's discussion and need no elaboration here. They form in large measure the basis for the growing popularity of commercial asphalt-rubber joint seals of the hot-poured type.

Whereas Stanton and co-workers approached the problem of advance preparation from the standpoint of first retarding gelation by use of an emulsifying agent, then promoting it by chemical treatment in the field, work in this laboratory concerned the possibility of transforming an asphalt-latex seal into one of the hot-poured type capable of being re-heated indefinitely, by actual removal of the water.

A typical case concerned an experimental joint sealed with a compound made in the following proportions:

1918 gm. SOA asphalt†
1082 gm. GR-S latex, 60 percent solids

*Michigan State Highway Department Research Laboratory.
†"soft oil asphalt" of 85 to 100 penetration.

These materials were blended by heating the asphalt to 200 F, adding the latex and stirring. Heating and stirring were continued for several hours, the temperature rising very slowly while the water was driven off. Eventually the water was removed, and the temperature of the material reached 425 F. The compound was poured into the joint at this temperature, slowly and with a single continuous pass.

A sample of the material poured at the same time the joint was sealed showed a 77 F, 100 gm., 5 sec. penetration of 0.67 cm., using a needle. (Similar penetration of a well-known commercial asphalt-rubber seal of the hot-poured type was 0.99 cm.) A year later, a sample of the seal was removed from the joint, the surface skimmed off, and a penetration specimen prepared. Penetration now was 0.55 cm., indicating that the material had become but slightly harder during a year in the joint. In spite of its hardness, this seal possessed excellent elasticity and adhesion after a year of service. The economic practicability of this method of driving water out of the seal by the application of heat has not been explored on a large scale.

Asphaltic oil-latex joint seals are relatively softer than asphalt-rubber seals of the hot-poured type. This, of course, may be considered a disadvantage from the standpoint of resistance to penetration by stones, and other foreign bodies. This fact prompted the substitution of SOA asphalt for asphaltic oil, resulting in a much lower penetration, and to the study of the use of various types of synthetic latices in addition to GR-S.

Synthetic latices tried included GR-S, Ameripol, Neoprene, Hycar and Revertex. In general, it was found that seals made with GR-S latex of at least 60 percent concentration of solids exhibited desired properties to the greatest extent, these properties including elasticity, adhesion, toughness and rapidity and ease of blending. This tends to substantiate Stanton's choice of GR-S latex, as specified by the California Division of Highways as a substitute for natural latex, and indicates that GR-S is probably the most suitable of available synthetics for this purpose.

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Petrography of Concrete Aggregate*

By ROGER RHOADES and R. C. MIELENZ†
Members American Concrete Institute

SYNOPSIS

Serviceability of a concrete aggregate depends upon the manner in which it joins with cement to determine the quality of concrete. Yet, standard acceptance tests do not measure properties which are directly responsible for performance of aggregates enclosed in concrete; new methods of aggregate investigation are needed. Experience shows that petrographic study can supply valuable information on a routine basis, and that, wherever possible, ordinary acceptance tests should be supplemented by examination by a petrographer familar also with problems of concrete. The significant properties of aggregates are discussed, and methods of petrographic study of aggregates are described.

An extensive bibliography is appended and referenced in the text for the benefit of readers, especially petrographers, wishing to explore further the concepts treated only briefly in this paper.

INTRODUCTION

Serviceability of concrete aggregates depends upon the strength and durability of the union which they establish with portland cement. Standard acceptance tests, currently applied independently to aggregate and to cement, do not permit satisfactory prediction of the quality of concrete. Such tests of aggregates may determine properties which make "good" or "poor" rock, but they contribute little to an interpretation of the rock's concrete-making properties. For example, tests on crushed quartz reveal how strong, impermeable, and durable it is, but they fail to indicate that it may nevertheless be an inferior aggregate. Durability or lack of durability, strength or weakness, or any other property of aggregate should concern the concrete engineer only to the extent that the property affects concrete. At this late date, service histories remain the only wholly reliable criteria of suitability. Conse-

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quently, a reorientation of acceptance testing is necessary; practicable tests must be designed which will determine the properties affecting quality of concrete. These properties are reviewed in this paper.

Properties of aggregates comprise 1) those of the individual particles, and 2) those characterizing the entire assemblage. The latter, which comprise gradation, moisture content, and bulk weight are easily determined and will not be considered here. Properties of individual aggregate particles originate in their mineralogic compositions, textures, and structures. These are petrographic properties, and they may be determined by petrographic methods. Petrography has long been used in aggregate selection[1-10]* but its use should be extended.

PETROGRAPHIC PROPERTIES OF AGGREGATE

Properties of aggregate are numerous and complex, and they originate in several ways. The original nature of the rock may control its concrete-making qualities. Thus, rocks containing gypsum are always somewhat soluble, opaline materials react chemically with alkaline solutions, and shales are usually soft and absorptive. Conversely, many properties are not directly related to the rock type but were induced by weathering, solution, saturation, or encrustation by ground waters, by fracturing and jointing related to the geologic history of the region, or by some other natural alteration. Thus, granites may be either hard or disintegrated, and sandstones may be either friable or tightly cemented, with resulting variations in strength and porosity. Therefore, petrographic study of concrete aggregates must be dual: 1) properties inherent to the rock must be interpreted in relation to 2) the natural alteration which it may have experienced.

The most important properties of concrete aggregate are discussed below. It must be noted that these properties are commonly independent of rock type since rocks of any given type may have widely different concrete-making qualities. Effects of secondary alterations, of which weathering is the most common, are important only insofar as the essential properties of the aggregate are changed; where significant, the possible effects of weathering are indicated.

Chemical reactivity

Few, if any, rocks or minerals are chemically inert while enclosed in portland cement. There are four main ways that an aggregate may be chemically deleterious. First, it may contain water-soluble constituents which can be leached, with attendant loss of strength and increase in porosity. Leached material may form unsightly efflorescence or indirectly lead to surface scaling[11, p. 56]. Second, soluble constituents or products of oxidation may retard or modify normal hydration of cement.

*Numbers in parentheses refer to bibliography at end of text.

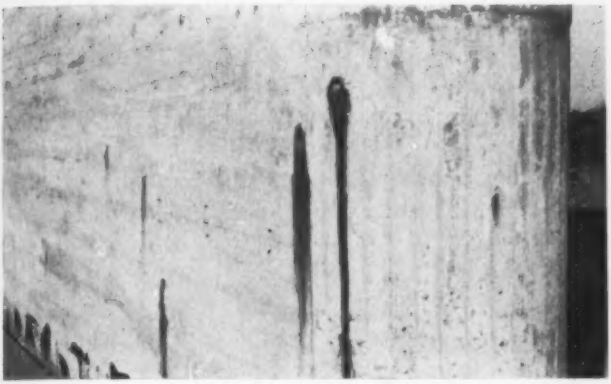

Fig. 1—Pop-outs and staining of concrete produced by oxidation of pyrite in particles of coarse aggregate.

Thus, oxidation of pyrite (FeS_2) in the aggregate resulted in pop-outs, local disintegration, and unsightly staining of the concrete at a dam in California[12] (Fig. 1).* Similar effects have been described by Litehiser[13]. Also in this category is the deleterious action of the mineral alunite ($K_2Al_6(OH)_{12}\ (SO_4)_4$) in mortar bars[14]. In one month the bars expanded significantly, cracked, and warped, probably as the result of sulfate attack on the cement, since secondary calcium sulfoaluminate was abundantly produced. Similar effects might be expected from other aggregates containing soluble sulfates[15]. Gypsum ($CaSO_4.2H_2O$) in excessive amounts is deleterious to concrete[16, pp. 188, 194].

Third, certain aggregates are attacked by high-alkali cements[9, 17]. The reactions lead to internal expansion, external cracking, and decreased elasticity and strength (Fig. 2 and 3). Rocks and minerals known to be susceptible to this kind of reaction are: opal; many volcanic rocks of acid to intermediate composition (high to medium silica content); glass (artificial or natural, excluding the basic types such as basaltic glass); chalcedonic rocks (including many cherts and flints); some phyllites; and tridymite[9,10,12,14,17,18,19,20,21,22]. Ameliorants are being sought but at present, where such rocks must be used, specification of low-alkali cement is imperative. Limitation of alkali content to below 0.6 per cent ($Na_20 + K_2O$, expressed as soda-equivalents) has avoided or significantly retarded deterioration, even though the aggregates are highly reactive[9,20,24].

Fourth, certain common minerals (e.g., zeolites) are capable of base-exchange. In this process, alkalies in these minerals are exchanged for calcium in solutions permeating the concrete[9]. Released alkalies may then attack susceptible aggregate particles, and efflorescence may form on or near the surface of the concrete. Studies by the Bureau of

*However, pyrite in aggregate is not always susceptible to such decomposition; many concrete pavements in and about Colorado Springs, Colorado, contain pyritiferous aggregates which show no signs of alteration after 19 years of service.

Reclamation and others suggest that such base exchange, involving a granodioritic aggregate, may account for efflorescence and surface scaling of concrete on the downstream face of a thin dam in southern California[12]. Tests conducted in the Bureau of Reclamation laboratories indicate that most natural aggregates will extract calcium from alkaline solutions. Rengade, Lhopitalier, and de Fontmagne[23] and, more recently, Hansen[18] have shown that some natural aggregates release Na_2O and K_2O when embedded in cement.

Porosity, permeability, and absorption

Pore characteristics of aggregate are important controls of chemical and physical stability, and they strongly influence the bond with cement. Pore characteristics significantly affect the strength of any material, and they determine absorption and permeability. As a result, they control both durability under freezing and thawing conditions, and rate of chemical alteration. Three characters of pores determine physical and chemical stability of aggregate. They are 1) absolute pore volume, 2) size of the pores, and 3) their continuity. These characters are reflections of the

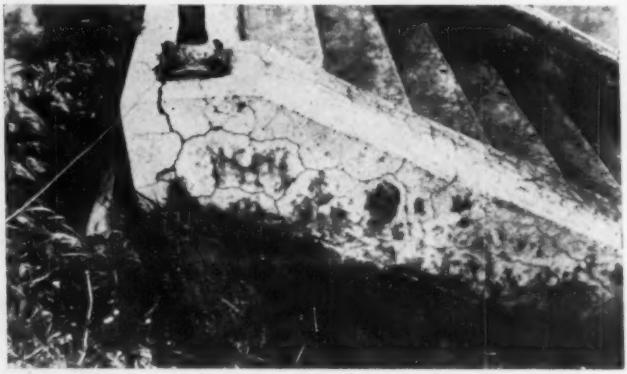

Fig. 2—Typical appearance of field concrete severely affected by reaction between aggregate and high-alkali cement. The concrete is 6½ years old.

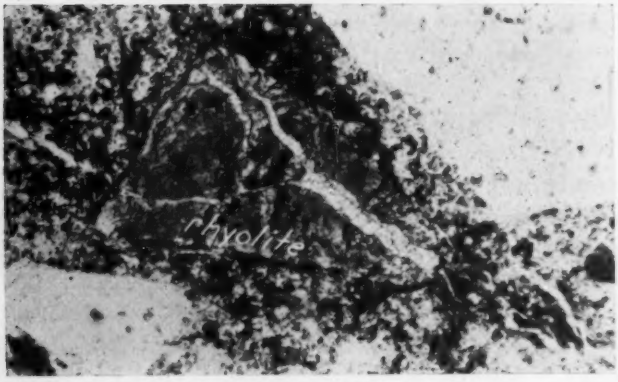

Fig. 3—Photomicrograph of concrete from Parker Dam, showing microfractures produced as a result of reaction between a particle of rhyolite and alkalies in cement.

Fig. 4—Fragment of quarried limestone in part unsound due to weathering action of ground waters. The white portion is highly absorptive, although the pores are microscopic in size.

texture of the rock, such as grain size, grain shapes, secondary cementation, and induration, and of structural features such as stratification and fractures. Original pore character may be changed by weathering, with resulting increases in porosity, permeability, and absorption; these changes may reduce durability and strength (Fig. 4). In spite of their importance these pore characters are not measured by any standard acceptance test.

Absorption is commonly used to predict freezing and thawing durability of aggregate. Investigators have variously obtained good to poor correlations between absorption and durability, the best relationships being found where similar rock types were used in the experiments. Where different kinds of rocks were used, only fair to poor correlations were derived. Wuerpel and Rexford[8], Cantrell and Campbell[25], and Sweet[26] obtained good relations between high absorptions and low durabilities of chert aggregates. But Hughes and Andersen[27], Garrity and Kriege[28], and others[29,30,31,32] did not find that durability consistently decreased as absorption of lithologically complex aggregates increased. Wuerpel[31] states that even with specifications which permit absorptions as high as 2 per cent, one-fourth of the rejected aggregates may be suitable for use in concrete.

The absorption test of bulk aggregates does not discriminate between absorption by few, highly porous particles or absorptions by many,

moderately porous particles; for this reason, absorptions of lithologically complex aggregates do not show quantitative relationships to soundness. Wray and Lichtefeld[32] have demonstrated how slowly moisture is regained after desiccation, the rates of absorption varying with porosity and permeability. Thus, a standardized period of immersion may give deceptively low absorption for some porous, unsound aggregate of low permeability. Garrity and Kriege[28] long ago pointed out the divergence of absorption by simple immersion from potential absorption; one sample tested absorbed 2.60 per cent in 18 hours at 70 F but 6.47 per cent in 5 hours in boiling water, whereas a second material showed an increase from 1.55 per cent to merely 1.69 per cent under the same conditions.

Freezing and thawing breakdown is controlled by degree of saturation. On this basis, the saturation ratios of Krueger[33] and Schurecht[34], and modifications of these ratios have been widely used in durability studies of building stone[11,33], brick[34], concrete[35,36], and concrete aggregate[26,29]. Saturation ratios are calculated as the ratio between short-time and long-time absorption, and are therefore functions of permeability. For concrete aggregate these ratios are significant where rocks of similar type are tested; but where a wide range of types is studied, the ratios and rates of disintegration are little related. However, permeability is a fundamental factor in resistance to freezing and thawing[37]. Sweet[26] found a relationship between depth of penetration of a dye solution and durability of cherts.

The effect of an aggregate particle upon freezing and thawing durability of concrete depends largely upon the ability of the particle to attain a high degree of saturation while it is enclosed by cement. Measurement of the imbibition of water while the particle is immersed in a water bath (such as by absorption itself or by the Schurecht or Kreuger ratios) does not necessarily indicate the capacity of aggregate to attain and retain a high degree of saturation in concrete. Whether or not water will enter into or be drawn from an aggregate particle while it is enclosed in concrete depends in considerable part upon the relative size of the pores in the particle, on the one hand, and those in the cement paste, on the other. Water moving by capillarity will not enter aggregate containing only large voids, even if these voids are interconnected and penetrable. On the other hand, small voids will be penetrated; and if these openings are smaller than those of the cement paste, the water will be preferentially drawn into them from the paste. During periods of hydration or drying, while water is being withdrawn from the interstices of the cement paste, water will be drawn by capillarity from aggregate particles containing only voids which are larger than those in the paste; but the last residuals of water remaining in even relatively dry concrete

may be concentrated in aggregate-voids smaller than those in the cement paste. Thus, rocks containing exceedingly small, interconnected voids are capable of attaining and retaining a high degree of saturation in concrete, and may be susceptible to disruption if repeated freezing occurs. Also, these materials will offer little opportunity for relief of hydrostatic pressures built up in the concrete by the progressive freezing postulated by Powers[37]. In view of the fact that most field concrete receives moisture through capillarity, these considerations are highly pertinent.

Extensive studies on building stones have shown a good relation between durability and content of voids smaller than 0.005 mm. (micropores), and that neither absolute porosity nor absorption is simply related to soundness[11, p. 33]. Moreover, rocks most commonly associated with freezing and thawing breakdown of concrete (such as argillaceous limestone, shale, and weathered chert) are characterized by exceedingly small voids. A soft, porous, and friable pumice with large voids, which was tested by the Bureau of Reclamation, showed 24.6 per cent absorption in 24 hours of immersion; yet concrete cylinders containing this pumice as both fine and coarse aggregate failed only after 350 cycles of freezing and thawing[38]. Many vesicular basalts, containing abundant large pores, have exhibited excellent durability in both field and laboratory concrete[12].

Bond and surface texture

Strength and permanence of bond between cement and aggregate are functions of surface texture and chemical stability of the aggregate. The integrity of original bond, however good, will be lost if certain chemical reactions, such as that between high-alkali cement and reactive aggregate, subsequently take place; but some types of chemical interaction may be beneficial in effecting a more intimate union. Surface rugosity of aggregate particles influences development of bond; but an even more important aspect of surface texture is the porosity, absorption, or permeability of the zone immediately underlying the surface. Penetration of the aggregate by cement slurry is conducive to good bond, but the porosity implied by very high penetrability may involve low tensile and shearing strengths of the aggregate, with net loss in strength of the concrete.

Surface texture depends upon original texture and structure of the rock, effect of weathering, and, in the case of natural gravels, upon processes of transportation and deposition which formed the deposit. The texture and structure of the rock are the most important. Certain minerals, such as quartz and feldspars, naturally break with smooth surfaces; and many common rocks, such as volcanic glasses and some cherts, do likewise because of dense structure. Effects of weathering are not

altogether undesirable inasmuch as slight surficial alteration will increase absorption and rugosities of the aggregate particles, and may thus increase integrity of bond with cement.

Hughes and Andersen[27] have shown that fine aggregates with very low absorptions generally develop less strong bonds and produce less durable mortars than those with slightly higher absorptions. Meissner[39] attributed low freezing and thawing durability of concrete bars containing crushed quartz aggregate to poor bond. The interrelation of bond and absorption may account in part for poor correlation between durability of concrete and absorption, inasmuch as strength of bond increases as absorption increases whereas durability of concrete tends to decrease as absorption increases. However, absorption of aggregates, as such, cannot be adduced as a reliable indication of good bonding characteristics, for capillaries of extremely small size may not permit penetration of the cement slurry into the aggregate particles, even though they may permit considerable penetration of water. Therefore, from the standpoints of both durability and bond, penetrable voids of very small size are the least desirable.

Internal fractures

Rock may be internally fractured but nevertheless present an external appearance of soundness. Fractures are common in rock types which are naturally "brittle," such as cherts and slates, but may exist in any rock either as a result of weathering or of earth movements, such as folding or faulting, which affected the region from which the rock was originally derived. Cracks may be microscopically small, but they increase absorption and porosity, and decrease strength and durability[8]. Moreover, they may become accentuated by temperature or stress changes within concrete.

Internal fractures are commonly produced or extended by crushing processes; and these changes generally result in higher losses in sodium sulfate and freezing and thawing soundness tests of the aggregate[2,12]. Original fractures, not produced in crushing, may commonly be detected visually by the presence of secondary materials such as iron oxide (common in slates and cherts) or stingers of clay (common in impure limestone), in cases where no openings are otherwise apparent. Although fractures may be tightly sealed through metamorphic processes taking place in the earth under conditions of high pressure and temperature, secondary materials deposited during weathering or related alterations do not prevent entrance of water into the rock.

Particle shape

High proportions of flat or elongated particles in concrete aggregate are objectionable since they decrease workability and hence necessitate the use of more sand, cement, and water[24, p. 63]. Also, they pack

poorly, reducing bulk weight[40] and decreasing compressive strength[41]. Moreover, flat particles tend to orient themselves horizontally in concrete, permitting the accumulation of bled water beneath them, which prevents the development of good bond on their lower surfaces[42].

Internal structure largely controls particle shape. Thus, the laminated structure of schists, slates, and shales commonly gives rise to slabby forms. Characteristically, rocks without planar structures such as massive varieties of granite or limestone break down to roughly equidimensional shapes.

Thermal expansivity

Two important aspects of thermal expansivity pertinent to the selection of concrete aggregate are: 1) the average coefficient for the entire aggregate and 2) divergence of the coefficients of the aggregate particles from that of the cement.

In designing structural elements, allowance frequently must be made for thermal volume changes of the concrete, and these volume changes are strongly influenced by the average expansivity of the aggregate[24].

Differences between the thermal expansivities of aggregate constituents and that of the cement have been held responsible for poor durability of concrete in service[43]. Aggregate particles with unusually low coefficients are of more importance than particles with exceptionally high coefficients, because few rocks have coefficients significantly greater than those of portland cements. With declining temperature, particles of very low expansivity may create high tensile stresses in the enclosing cement, and disruption of the concrete can result[43].

Although expansivity of aggregate affects durability of concrete, the significance of different expansivities of different components of an aggregate is difficult, if not impossible, to evaluate in most instances. Measurement of the coefficient of expansion of an aggregate or of its constituents is not practicable on a routine basis. Although petrographic identification of constituents will assist in estimation of expansivity, similar rocks may have widely different coefficients (Table 1); in addition, many rocks and minerals have different coefficients in different directions[39,44,45,46]. Moreover, the effects of differences between the expansivities of aggregate and cement, unless very large, are commonly obscured by other variations (for example, aggregates with different coefficients of expansion usually also have different chemical compositions, surface textures, pore characteristics, etc.). For structural reasons, concrete possessing a low over-all coefficient of expansion may in specific instances be so desirable as to outweigh the advantages of matching the coefficients of the aggregate constituents with that of the cement.

TABLE 1—THERMAL COEFFICIENTS OF EXPANSION OF ROCKS*, CONCRETES, AND PORTLAND CEMENTS

(Expansion per degree Fahrenheit, in range—4 to 212 F)

Materials	Number of Specimens	Observed range in Mean Linear Thermal Expansion Coefficient $\Delta L x 10^6 L \Delta T$	References
Granites and Rhyolites	27	1.0 to 6.6	45,46,49,51
Diorites and Andesites	17	2.3 to 5.7	39,45,46,49
Gabbros, Basalts, Diabases	15	2.0 to 5.4	45,46,49,51,52
Sandstones	24	2.4 to 7.7	45,46,49,50,52
Quartzites	20	3.9 to 7.3	45,46,49,51,52
Dolomites	7	3.7 to 4.8	39,45,46
Limestones	65	0.5 to 6.8	39,45,46,49,50,51
Siliceous Limestones	6	2.0 to 5.5	45,46
Cherts	49	4.1 to 7.3	45,46,49,51,52
Marbles	29	0.6 to 8.9	43,47,48,49
Slates and Argillites	5	4.5 to 4.8	45,46,49
Portland Cements, neat	10	5.9 to 9.0	24,51
Concretes	27	3.6 to 6.8	24

*All coefficients of rocks were obtained on dry specimens.

Surface coatings

Surface coatings are generally detrimental because: 1) the strength of bond between the rock and coating is generally less than the internal strength of the rock, and may be very low, so that the union between the rock and the cement is weakened[15]; 2) many natural coatings contain substances susceptible to reaction with alkalies in cement (for example, opaline silica); thus, aggregates which are inherently innocuous may become deleterious because of the surface coating[9]; 3) coatings may contain salts of sodium or potassium which, if released, may participate in deleterious chemical reactions with susceptible constituents of the aggregate[9], or coatings of soluble salts such as gypsum ($CaSO_4.2H_2O$) may influence the course of cement hydration[16]; and 4) if coatings are broken free during handling, and the residue not subsequently removed by washing, water requirement may be increased[53]. In some instances, coatings may be essentially inert and so strongly bonded as to be innocuous, or they may actually increase the strength of the bond between aggregate and cement.

Properties affecting drying shrinkage

Carlson[54] has suggested that harsh aggregates, by increasing water requirement for given workability, may contribute to drying shrinkage, and that this shrinkage will be increased by particles which swell and contract upon wetting and drying. He tentatively concluded "that compressibility is the most important single property of aggregate governing concrete shrinkage," shrinkage increasing with increasing compressibility. He suggests that low shrinkage induced by more rigid

aggregates probably results in a concrete more susceptible to breakdown by freezing and thawing through the formation of minute cracks in the cement paste.

Several aggregates containing high proportions of very coarse-grained, relatively unaltered granites have been found to produce concrete with low flexural strength and low resistance to breakdown under freezing and thawing, heating and cooling, and wetting and drying conditions [12,55]. These properties may result either from poor initial bond of the cement to the smooth, nonpermeable rock surfaces, from rupture of original bond due to differences in thermal coefficients among the several minerals of the granite or between the granite and the cement matrix, or, as may be implied by Carlson's experiments[54], from the formation of abundant minute cracks in the cement paste adjacent to the granite particles which have low compressibility.

Strength and elasticity

Although unusually soft aggregate particles reduce strength and elasticity of concrete, even under circumstances not involving durability, most rocks have considerably greater strengths and moduli of elasticity than even highest quality concrete. For example, average crushing strengths of many granitic rocks, gneisses, quartzites, marbles, sandstones, limestones, slates, and serpentines range from 10,500 to 28,700 pounds per square inch[44], whereas ultimate compressive strengths on the order of 5,000 to 8,000 pounds per square inch are considered quite satisfactory for most concretes. The strength and elasticity of sound aggregate are rarely factors controlling suitability. The strength and elasticity of concrete, containing sound aggregate and not affected by cement-aggregate reaction, are controlled largely by the bond established between cement and aggregate, the quality of cement, mix design, and placing and curing techniques.

Specific gravity

Specific gravity of aggregate influences the unit weight of concrete, and thus is of first importance under some conditions of service, such as in gravity sections of dams where maximum weights of concrete may be desired, or in flooring, where minimum weights are necessary[24]. In these circumstances specific gravity may be a determining factor in the selection of an aggregate. However, in general, the specific gravity of aggregate, other than certain artificial types, ranges between comparatively narrow limits, and the unit weight of concrete is not critical for most purposes. Of local importance are extremely light, unsound particles which segregate toward the surface, and thus contribute to low abrasion resistance and surface durability of the concrete[56].

Specific gravity determinations on aggregates are commonly utilized as an indirect measure of soundness, the conception being that sound-

ness will decrease as the specific gravity decreases. Specific gravity is a function of the composition of the rock and its porosity; thus a rock with high absolute specific gravity will have a considerably higher porosity at a given apparent specific gravity than a rock with a low absolute specific gravity. For example, with an apparent specific gravity of 2.40, pure chert or quartzite would have a porosity of 9.4 per cent; whereas, with the same specific gravity pure dolomite would have a porosity of 14 to 17 per cent. Fear of low specific gravity is based upon fear of high porosity. Thus, the discussion in the section dealing with porosity, permeability, and absorption is pertinent in this connection.

The relationship between specific gravity and durability has commonly been considered in the selection of chert aggregates. Wuerpel and Rexford[8] observed that all chert fragments related to pop-outs in field concretes possessed apparent specific gravities (A.S.T.M. designation: bulk specific gravity, saturated-surface-dry) less than 2.40. On this basis and in view of experimental data, these authors recommend that cherts with less than this specific gravity be considered unsound. Cantrill and Campbell[25] have found that chert gravels from the Tennessee and Cumberland Rivers with apparent specific gravities (A.S.T.M. designation: bulk specific gravity) less than 2.33 and 2.44, respectively, are unsound in service. Sweet[26] concludes that unsound Indiana cherts are characterized by specific gravities (A.S.T.M. designation: bulk specific gravity, saturated-surface-dry) less than 2.50; but he mentions an unsound Ohio chert with an apparent specific gravity of 2.51, thus emphasizing the necessity for restricting a given specification to aggregates of one specific lithologic variety. Wuerpel and Rexford[8] point out that even if restricted to chert aggregates, their specification limit will exclude only 70 to 90 per cent of the unsound materials, largely because of the presence of impurities with high specific gravities. Walker[57] records the occurrence of 9.1 to 56.0 per cent by weight of chert particles with specific gravities less than 2.40 in four gravels supplied by long-established commercial producers. Lang and Hughes[58] in tests of aggregates of different lithologic types found no correlation between freezing and thawing durability and specific gravity.

Specific heat and thermal conductivity

The Bureau of Reclamation[59] has shown that thermal conductivity and specific heat of aggregate exert a strong influence upon the transmission of heat through and changes of temperatures in concrete. Where the following rock types were used as coarse aggregate, other conditions being identical, the specific heats of the concrete increased in this order: quartzite, granite, rhyolite, limestone, basalt, dolomite. The thermal conductivities of these concretes increased in the following order of aggregates: basalt, rhyolite, granite, limestone, dolomite, quartzite.

Calculations of diffusivity based upon the specific heats, densities, and thermal conductivities indicate that the different concretes will have significantly different rates of temperature change under equivalent conditions; the rates of temperature change increasing in the following order of aggregates: basalt, rhyolite, granite, dolomite, limestone, quartzite. Rates of heat transmission and temperature change must be considered wherever external or internal sources of heat are expected to produce significant thermal changes in volume.

SELECTION OF AGGREGATES

Standard acceptance tests of aggregates yield only meager information concerning the fundamental physical properties discussed above, and they largely ignore the pertinent chemical factors. If an aggregate fails these tests, the cause of failure may still be unknown. But the test data are amenable to quantitative evaluation in terms of a certain number of cycles, or weight loss, or some other definite quantity. These figures may be used arbitrarily in defining specification limits, although it is recognized that results of any given tests of this sort cannot be reliably correlated with service records[30]. Indeed, some aggregates which consistently fail the standard tests for physical soundness possess excellent service histories.

A test which will quickly reveal the reactivity of proposed aggregates with alkalies in cement is badly needed. Many tests have been suggested, but all are unsatisfactory. The degree of reactivity can be determined by measurements of expansion or strength decline of mortar bars containing high-alkali cement; but the several months commonly required for conclusive results, limit the practicality of the method [9,17,18,20,21,60]. Stanton[9,60] has suggested tests which are applicable to aggregates containing highly reactive opal, but they are inconclusive for less reactive materials. Leach tests[9,21,61,62,63] based on the chemical effects of alkaline solutions on aggregates are not yet perfected as indicators of potential cement-aggregate reaction. An etch test proposed by Parsons and Insley[64] is a modification of the leach test, involving microscopic examination of polished sections which have been treated with alkaline solutions; this test does not measure the particular kind of reactivity which causes deleterious expansions of mortars and concretes. Use of ultra-violet light and dyes for detection of cement-aggregate reaction is inadequate and such methods are unsatisfactory for predicting reactivity of aggregates[10,12,60].

Petrographic examination is primarily a visual appraisal of aggregate; any weaknesses are described as to character and extent; in a mixed aggregate any desirable or undesirable properties are identified with particular constituents; and relative qualities of different size

grades are determined. The method does not lend itself to definition of specification limits because information so obtained is the result of subjective appraisal by the petrographer, and can be reduced to a numerical quantity only through his personal interpretation. On the other hand, difficulties attending the formulation of specifications based upon results of standard soundness tests have likewise been recognized[8].

In the Bureau of Reclamation procedure of petrographic analysis, aggregates are examined visually, and representative fractions are segregated into appropriate lithologic groups, based upon three considerations: 1) petrographic identity, 2) physical quality, and 3) chemical stability. For example, particles of granite may be placed in several categories depending upon the effects of varying degrees of weathering, or limestones may be classified as chemically innocuous or deleterious because of the absence or presence of an unstable minor constituent. In most instances, the study of natural gravels is satisfactory if only the 1½- to ¾-in. and ¾- to ⅜-in. fractions are analyzed in detail, and the other fractions are then qualitatively compared with them. Sands are examined in detail under binocular and petrographic microscopes. Where necessary, supplemental partial analyses, involving the segregation of physically or chemically deleterious particles, are made of any size-fraction of the aggregate. Typical summarization of a Bureau of Reclamation petrographic analysis is shown in Tables 2 and 3. These analytical data are always accompanied by a memorandum describing pertinent features of the aggregate and including specific recommendations regarding use of the aggregate.

The usual physical tests are applied to all aggregates investigated by the Bureau of Reclamation; and petrographic examination is used to supplement and interpret the results of these tests, or independently for preliminary estimates of quality. For example, in the sodium-sulfate soundness test (A.S.T.M. Designation: C 88-41T)[65], breakdown of platy particles may not be indicated by loss through the screens since, if the fracturing takes place by splitting parallel to the flat surfaces, no effective decrease in diameter is produced. This situation was encountered in tests of a phyllite aggregate proposed for use at Fontana Dam[12]. Low specific gravity of natural aggregates is commonly used as an indication of porous, soft, or deeply weathered materials. However, several natural aggregates having specific gravities of 2.44 to 2.56 from the vicinity of Provo, Utah, possess excellent service histories; these aggregates are composed predominantly of firm or hard but porous sandstones, and were quickly found by petrographic examination to contain merely insignificant proportions of clayey or friable materials,

TABLE 2—PETROGRAPHIC ANALYSIS OF A COARSE AGGREGATE

Typical tabular summary of a Bureau of Reclamation petrographic analysis of coarse aggregate.

Rock Types	Percentages by Weight			Quality	
	1½-¾ in.	¾-⅜ in.		Chemical	Physical
Granites	1.7	9.0	Slightly to moderately weathered, coarse-grained	Innocuous	Satisfactory
Rhyolite Porphyries	1.5	1.1	Slightly to moderately weathered, aphanitic, porphyritic	Innocuous	Satisfactory
Andesite Porphyries	29.5	17.2	Slightly to moderately weathered, aphanitic to fine-grained	Innocuous	Satisfactory
Weathered Andesite Porphyries	11.9	10.6	Moderately weathered, fractured, firm to soft	Innocuous	Fair
Andesites	9.8	3.9	Slightly to moderately weathered, aphanitic to glassy	Deleterious	Satisfactory
Basalts	18.1	29.3	Slightly to moderately weathered, aphanitic to glassy	Innocuous	Satisfactory
Weathered Basalts	21.9	21.8	Moderately weathered, fractured, firm to soft	Innocuous	Fair
Meta-Sandstones and Quartzites	5.6	7.1	Hard, massive to slightly schistose, fine-grained	Innocuous	Satisfactory

*See notes for Table 3.

TABLE 3—PETROGRAPHIC ANALYSIS OF A COARSE AGGREGATE

Typical tabular summary of the quality of coarse aggregate, based upon petrographic analysis.

	Degrees of Quality*	Percentages by Weight	
		1½-¾ in.	¾-⅜ in.
Chemical	Innocuous	90.2	96.1
	Deleterious (with high-alkali cements)	9.8	3.9
Physical	Good	....	....
	Satisfactory	66.2	67.6
	Fair	33.8	32.4
	Poor	....	....

*Explanation of terms used to describe the quality of aggregates.

Innocuous and deleterious

The classifications "innocuous" and "deleterious" refer to the chemical reactivity of aggregate constituents, particularly their reactivity with alkalies in cement. "Deleterious" materials produce adverse effects on concrete due to chemical reactions which take place subsequent to hardening of the cement. Substances designated as "deleterious" by reason of susceptibility to attack by alkalies in cement are non-deleterious if used with cements containing less than 0.60 per cent total alkalies (Na_2O plus K_2O, expressed as soda-equivalents). "Innocuous" constituents do not participate in chemical reactions harmful to concrete.

Good, satisfactory, fair, and poor

Four classifications are employed to describe the contribution made by the physical properties of the aggregate constituents to the quality of the concrete, as follows:

Good aggregate constituents contribute to superior strength, abrasive resistance, and durability under any climatic conditions.

Satisfactory aggregate constituents contribute to high or moderate strength, abrasive resistance, and durability of concrete under any climatic conditions.

Fair aggregate constituents contribute to moderate strength and abrasive resistance of concrete, but under rigorous climatic conditions they contribute to physical breakdown of concrete.

Poor aggregate constituents contribute to low strength and abrasive resistance of concrete under any climatic conditions, and cause physical breakdown of concrete under rigorous climatic conditions.

and so were recommended for use in spite of the fact that their specific gravities were below specification limits[12].

Periodic petrographic examination of a deposit during exploitation will disclose significant vertical or lateral variations. For example, gravel in the lower course of the San Joaquin River in the Sierra Nevada typically contains a highly reactive andesite[9]; but certain deposits, or at least certain parts of some deposits, are virtually free from these andesites. The aggregate produced from these latter deposits might safely be combined with cements of moderately high alkali content. But, unless low-alkali cements are used, extreme care must be exercised in order that the limits of the andesite-free deposits are not transgressed. Again, near Pasco, Washington, many satisfactory, basaltic gravel

deposits are underlain by deeply weathered, physically unsound gravels of similar composition[12]. Investigation of gravel deposits in this region must reveal the presence of these weathered zones and locate their limits; exploitation must be conducted so as to prevent admixture of the weathered materials with satisfactory aggregate.

Petrographic examination may be fruitfully employed in the search for new deposits of natural aggregate. The rapidity with which reliable examinations and comparisons can be made permits the evaluation of aggregate quality even when only limited time is available, or the inaccessibility of laboratory equipment renders impracticable the application of most standard tests.

CONCLUDING STATEMENT

Acceptance testing of concrete aggregates is unsatisfactory at the present time, since none of the standard tests yield information which properly correlates with service of concrete; the whole concept of aggregate testing needs revision and reorientation, so that these deficiences will be overcome. The first requirements are tests to evaluate 1) pore characteristics, 2) bonding characteristics (surface textures) and 3) chemical stability, particularly with regard to "cement-aggregate" reaction.

Petrographic methods represent a useful approach toward these and other objectives. The principle values of these methods are: 1) unique, fundamental information not derived from standard tests is obtained; 2) quick, preliminary estimates of quality can be made; 3) they permit interpretation and amplification of the results of other acceptance tests; 4) petrographic identification of chemically deleterious substances is the only reliable means by which chemical instability of new aggregates can be quickly estimated at the present time; 5) new aggregates may be compared with other aggregates for which service data are available; and 6) petrographic methods can be used to advantage in exploration for new sources and in exploitation of partially developed deposits.

The possibility of comparing new with previously tested or used aggregates is of great importance. Tests or service histories upon aggregates which are not completely described are of limited value, since they indicate the qualities of the one aggregate and no other. If aggregates are completely identified from a petrographic standpoint, results of tests or service are then of general interest, and can be utilized in selection of new aggregates of similar lithologic type, either in the same or in distant regions. It is unfortunate that so many aggregates are meticulously tested but yet are described in the literature as "crushed stone," "river gravel," "sand," or vaguely, as "trap," and so on.

Petrographic study of aggregates is not a panacea by which all the ills inherent to selection of concrete aggregates can be cured, but it

does afford a means of evaluating significant factors not otherwise discovered. It is therefore recommended that, wherever possible, the routine testing methods be supplemented with examination by a petrographer familiar with problems of concrete.

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To facilitate selective distribution, **separate prints** of this title (42-24) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946.

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Entrained Air In Concrete

A SYMPOSIUM: contributions by:

W. A. CORDON
L. E. ANDREWS
ALEXANDER FOSTER, JR.
STANTON WALKER AND
DELMAR L. BLOEM
HENRY L. KENNEDY
EDWARD W. SCRIPTURE, JR.
A. T. GOLDBECK
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STANTON WALKER
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S. W. BENHAM
W. F. KELLERMANN
W. H. HERMAN
B. H. WAIT

FOREWORD

Entrained air in concrete provided a full-packed session at the Institute's 42nd annual convention in Buffalo in February; with one paper already published on laboratory work on the subject by the Corps of Engineers, U. S. Army* and 14 other contributions, several of them presented only in their highlights to be within the allotted 10-minute limitations of the session. These other contributions, most of them reporting on various aspects of the general theme, are here presented in a collected group.†

Concretes containing air-entraining agents are being extensively employed in present-day construction, and, as entrained air alters many of the basic properties of concrete mixes, such as water requirements, sand requirements, workability, etc., a series of trial mixes was made in the laboratories of the Bureau of Reclamation to establish procedures for designing and adjusting concrete mixes containing entrained air. W. A. Cordon (p. 605) writes about the useful data that have been established from this set of tests which should facilitate the adjustment and design of air-entrained concrete and also establish a basis for further investigation.

L. E. Andrews, in his paper (p. 621) describes preliminary studies and current practice in the northeastern states in the reduction of frost action

*"Laboratory studies of concrete containing air-entraining admixtures", by Charles E. Wuerpel, ACI JOURNAL, Feb. 1946.

†A separate publication, to be announced, will contain the symposium papers and other recent contributions to the present theme.

on concrete paving and other projects (roads, streets, airports, pavements and hangers, bridges and buildings); mix-design methods for determining air content and field control. Specifications tend to limit air content to 3 to 6 per cent rather than to fix the amount of air-entraining agents used.

Alexander Foster, Jr. reports studies by the Warner Co. over a 2½-year period with central mixed concrete using air-entraining cement. Following experimental work, more than 30,000 cu. yd. of air-entraining concrete, chiefly of low-slump, plastic mix, were used for pavements and highway work. Studies included the effect of truck mixer or agitator action, closed-drum and open-top equipment, on hauls up to 45 minutes. No significant differences in slump or air entrainment (2½ to 3 per cent) were found. The chief problem of the ready-mixed concrete people appears to be the added storage requirements necessary to meet specifications permitting no admixtures as well as those demanding air-entrained concrete (p. 625).

Stanton Walker and Delmar L. Bloem (p. 629) write of problems of air entrainment in concrete that have been particularly interesting to the ready-mixed concrete industry which has to meet a wide variety of specification requirements. Their paper describes exploratory studies in the research laboratory of the National Ready Mixed Concrete Association. Data are reported on the effects of entrained air on compressive strength and mixing water requirements. Other factors considered are: mixing time, grading of aggregate, temperature, ratio of sodium hydroxide to Vinsol resin, comparison of fresh and hardened concrete and air content at different depths of concrete.

Henry L. Kennedy (p. 641) reports studies of the homogeneity of air-entraining concrete by means of a test for resistance to abrasion. An air-entraining agent was used with no added accelerators, dispersing agents or gas forming materials. "Since air entrainment has a tendency to eliminate bleeding of concrete it is only reasonable to believe that such concretes are more homogeneous . . . . the top surface (of a slab) would have the same strength and density characteristics as the bottom surface." Plotted data show that there is progressive increase in abrasion loss in specimens as air content increases beyond 6 per cent.

Edward W. Scripture, Jr. (p. 645) discusses methods for and mechanisms of air entrainment in concrete mixes. Methods include use of aluminum and hydrogen peroxide for entrainment of hydrogen or oxygen, respectively; use of cement dispersing agents; perhaps protective colloids. Data are reported to record the relation of air content to durability as determined by freezing and thawing.

A. T. Goldbeck (p. 649) writes that a limestone sand which had been used with indifferent success in pavements and other structures, prompted

a series of tests by National Crushed Stone Association, to improve workability and durability of concrete in which this aggregate was used. Results with and without Vinsol resin as an air-entraining agent were favorable to the use of the admixture. Data reported include materials, mix design, freezing and thawing tests.

W. H. Klein and Stanton Walker (p. 657) in their paper describe, with illustrations, the Klein Air Meter and a test procedure and calculation of air content based on the application of Boyle's law which has advantages over the methods now in use to determine percentage of air entrained in concrete. The paper is based on results of tests made by the Pennsylvania-Dixie Cement Corp. and the National Ready Mixed Concrete Association.

R. R. Kaufman (p. 669) describes an automatic dispensing apparatus designed for getting an admixture into the concrete mix at the mixer in any constituent of the mix, with all the accuracy desirable.

E. M. Brickett (p. 673) describes several devices for accurate measurement of liquid admixtures as introduced into the concrete batch at the mixer with special reference to air-entraining agents as used in ready-mix and concrete products plants and on paving jobs. Since the quantity of the solution is relatively small, accuracy is important for uniform results.

S. W. Benham (p. 677) reports the procedures and results of a method for field and laboratory for determining the amount of entrained air in a concrete mix.

W. F. Kellermann (p. 681) presents a part of the results obtained from an investigation of the durability of concrete by the Public Roads Administration using blends of portland cements with natural cements (86 per cent and 14 per cent by weight, respectively) and Vinsol resin-treated cements. Results presented in this contribution have a bearing on resistance to freezing and thawing tests, especially because of the unusual results of a prolonged interruption of the daily freezing and thawing cycle.

W. H. Herman (p. 689) relates the experiences of the Pennsylvania Department of Highways with air-entraining concrete in which 331,555 bbl. of normal strength portland cement containing Vinsol resin were used since 1940. The Pennsylvania Department's attitude on the subject of air entrainment is characterized by more concern with the particular admixture used than with the percentage of air entrainment and such use was inspired by difficulties with finer ground cements which prompted seeking an additive to improve pavement durability.

B. H. Wait (p. 697) suggests that air entrainment by admixtures is not the only method feasible for combatting disintegration due to frost

action. Results are reported on numerous paving jobs in northeastern United States in support of the use of portland-natural cement blend as a means of reducing disintegration from frost action, where the air entrained averaged about 1 per cent only. Results were satisfactory and the weight of the concrete was higher than for straight portland-Vinsol resin mixes.

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Entrained Air—A Factor in the Design of Concrete Mixes*

By W. A. CORDON†
Member American Concrete Institute

SYNOPSIS

Concretes containing air-entraining agents are being extensively employed in present day construction, and as entrained air alters many of the basic properties of concrete mixes such as water requirement, sand requirement, workability, etc., a series of 102 trial mixes was made to establish procedures for designing and adjusting concrete mixes containing entrained air. Useful data have been established from this set of tests which should facilitate the adjustment and design of air-entrained concrete and also establish a basis for further investigation.

INTRODUCTION

Many of the empirical rules and formulae heretofore used in concrete mix design must now be modified to allow for changes resulting from the entrainment of air. It has been established among other things that the addition of air to concrete (a) reduces the water requirement, (b) reduces the sand requirement, (c) increases the durability, (d) decreases the strength at any given W/C ratio and (e) improves the workability.

A series of 102 mixes was made in the Materials Laboratories of the Bureau of Reclamation for the purpose of establishing procedures which could be applied to the design of average concrete mixes containing nominal amounts of entrained air. The results of this series of mixes, which include aggregates from four widely separated sources, screened to 1½-in. maximum size, cements from three different mills, and two different air-entraining agents, are presented in this paper.

The results of the tests show the quantity of air entrained in a given concrete mix is dependent on a number of variables including (a) type

*Presented at Air Entrainment Session, ACI Convention, Feb. 19, 1946.
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and grading of aggregate, (b) amount and brand of cement, (c) consistency of mix, (d) type of mixing, (e) mixing time, and, of course, (f) type and amount of air-entraining agent. It is not within the scope of this paper to discuss these various ramifications of air-entraining properties in detail. The discussion is therefore purposely limited to the properties affecting basic mix design.

It is indicated from analysis of results obtained that the tests made were sufficiently conclusive in the range of normal air contents of from about 1 to 8 per cent for the materials used and for the relationships covered by the tests. The curves presented were fitted by the method of least squares and were found to be "highly significant" statistically[1]* indicating that the results would be reproduced 99 times out of 100 if comparable materials and methods were employed.

CONCLUSIONS

1. ACI Standard Recommended Practice for the Design of Concrete Mixes[2] can be readily modified to include concrete mixes containing nominal amounts of entrained air.

2. The water requirement of an average concrete mix is reduced approximately 6.0 lb. per cu. yd. with rounded aggregate and 8.0 lb. per cu. yd. with angular aggregate for each per cent of air entrained. (Fig. 1, 2, and 2A). This reduction in water at constant 0.55 W/C ratio reduces the cement content 11 lb. and 14.5 lb. per cu. yd., for natural and angular aggregate, respectively, for each per cent of air entrained.

3. The reduction in water content for each per cent increase in air is about the same for any given water-cement ratio, slump, or brand of cement. (Fig. 2A and 3).

4. Sand content, by weight of total aggregate, may be reduced by one per cent for each per cent increase in entrained air up to about 8 per cent, without any appreciable change in workability or slump. (Fig. 4). (Each per cent reduction in sand permits a 2.5 lb. per cu. yd. reduction in water in addition to that mentioned in conclusion 2 above)[2].

5. The workability of a concrete mix is improved as the amount of entrained air is increased. (Fig. 5).

6. Where the water-cement ratio is held constant the compressive strength of concrete will be reduced approximately 200 psi for each per cent of air entrained in the fresh concrete by Vinsol resin. (This value does not apply where accelerators are used in combination with the air-entraining agent). (Fig. 6 and 6A).

7. Where the cement content is held constant the change in strength ranges from practically no change for concretes of low cement content to a reduction of around 200 psi for each per cent of air entrained for concretes of high cement content. (Fig. 6A).

*The numbers in parenthesis refer to the references appended herto.

8. The modulus of elasticity of an average concrete having the same water-cement ratio and the same aggregate grading, will be reduced approximately 105,000 psi for each per cent of air entrained in the fresh concrete. (Fig. 7).

9. Poisson's ratio is not significantly affected by the entrainment of air.

10. The quantity of agent required to entrain a given amount of air decreases as the slump of the concrete is increased. (Fig. 8).

11. Crushed and angular aggregates will entrain more air than well rounded aggregates. (Fig. 9).

12. The quantity of agent required to entrain a given amount of air is influenced by the brand of cement; also a smaller amount of agent is required for lean mixes than for rich mixes. (Fig. 10).

DISCUSSION OF TEST RESULTS

The amount of water required to produce concrete of a desired slump is an important consideration in the design and adjustment of concrete mixes and as entrained air has a marked effect on the water requirements of a mix, considerable emphasis was placed on the determination of this effect. Fig. 1 shows that an average reduction of 6.4 lb. of water per cu. yd. of concrete results from each per cent increase in air content. Fig. 2, 2A, 2B, and 3 show a breakdown of the points plotted on Fig. 1. These figures show that this decrease in water for each increase in air is uniform for the same slump and water-cement ratio, and that the decrease in water for each increase in air will be slightly different for different aggregates and brands of cement. The most noticeable example of the difference caused by type of aggregate is shown in Fig. 2 where a reduction of about 6 lb. for each per cent of entrained air was obtained for rounded aggregate and 8 lb. for angular aggregate.

It should be noted that the values established in the foregoing discussion do not include the reduction in water content resulting from reductions in the sand percentage which may be realized because of increased workability caused by the entrained air. Table 8 of the Standard Recommended Practice for the Design of Concrete Mixes[2] shows a reduction in water of approximately 2.5 lb. per cu. yd. of concrete for each per cent decrease in sand. Therefore, if the workability of the concrete is held constant by reducing the percentage of sand by one for each per cent of air entrained as shown in Fig. 4, the total reduction in water content would average approximately 9.0 lb. for each per cent of air entrained. (Fig. 2B).

Effect of entrained air on the workability of concrete mixes

Entrained air improves the workability by increasing the slump, thereby permitting a reduction in water content, as indicated in Fig. 1. Where the slump is held constant through reductions in water and

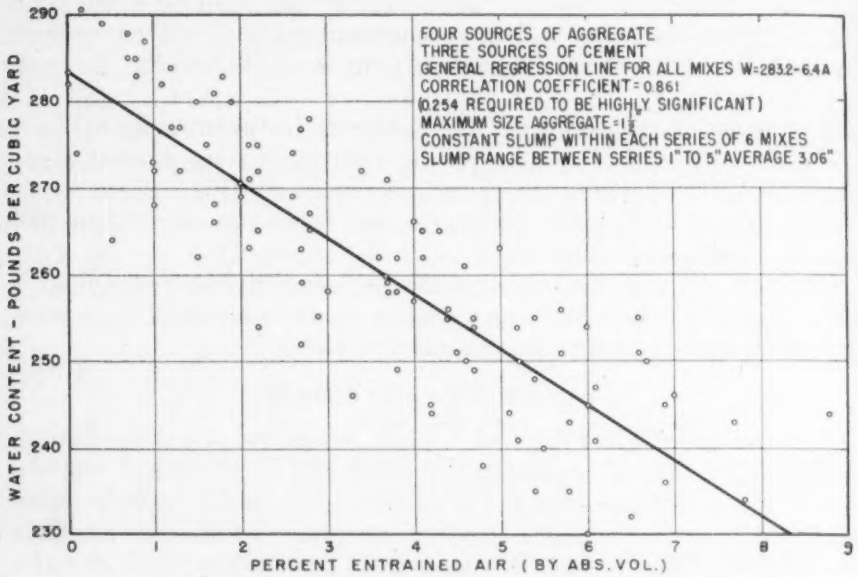


Fig. 1—An average reduction of 6.4 lbs. of water per cu. yd. of concrete results from each one percent increase in air content

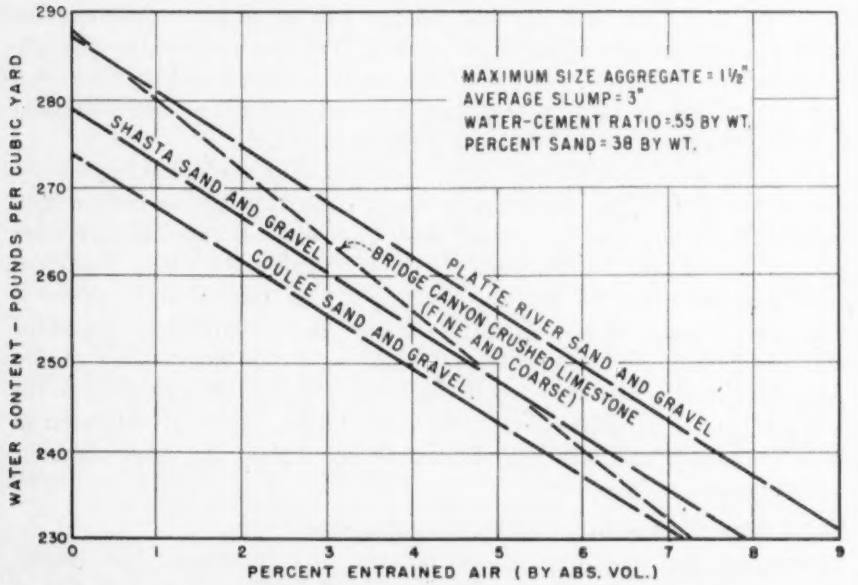


Fig. 2—The decrease in water content due to air entrainment is approximately the same for different natural aggregates, but is greater for crushed aggregate

cement as in Fig. 5, the workability was found to be better than that of the concrete of lower air content, as measured by "Power's Remolding Apparatus"[4]. Fig. 5 shows that the workability of a mix made with natural aggregates and containing 3 per cent air with a 1½-in. slump is about equal to that of a mix containing 1 per cent air with a 3-in. slump, even though the cement content has been reduced.

Decrease in strength with the entrainment of air

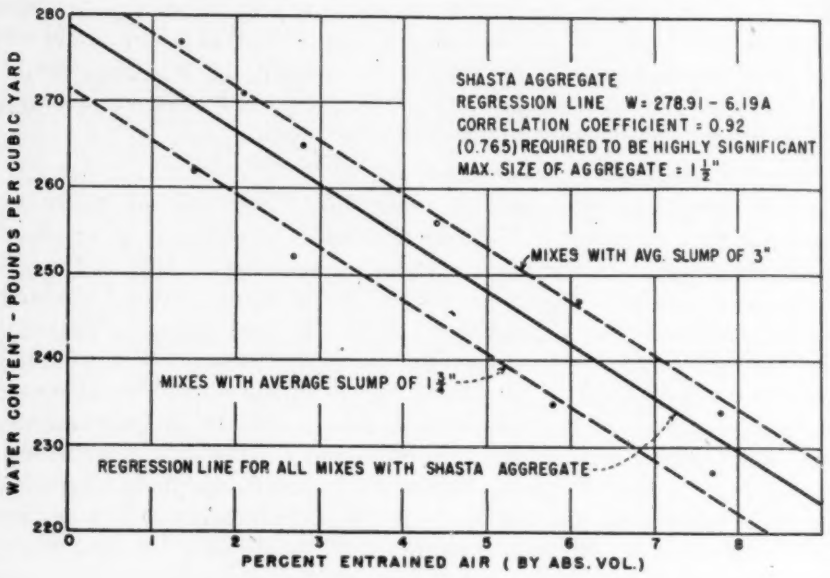
The strength of concrete as shown in Fig. 6 decreases uniformly with increases in air content of the fresh concrete. At constant water-cement ratio by weight the decrease in strength amounted to 195 psi (about 5 per cent average) for each per cent of air entrained. Also, where the W/C is held constant, Fig. 6A shows about the same reduction in strength for each per cent of air entrained for the higher constant W/C of 0.65 as for the lower constant W/C of 0.45. However, where the cement content is held constant, and the water-cement ratio is reduced through reduction in water content as a result of the entrained air, the change in strength for each per cent of air is not constant and ranges from a slight increase for mixes of low cement contents to about 200 psi reduction for each per cent of air for mixes of high cement content. It should be noted that there is little advantage in maintaining the cement content constant when using more than 6 sacks per yard, since the reduction in strength as the air is increased is about the same as that for constant W/C.

Decrease in modulus of elasticity with entrainment of air

Although there was close agreement on the reduction of the modulus of elasticity within each group of tests containing the same aggregate, Fig. 7 shows the marked difference between two different types of aggregate. The slopes of the regression lines correspond with the average of all tests, however, and at the same water-cement ratio a loss of approximately 105,000 psi static modulus of elasticity for each per cent of air entrained can be expected for average concrete mixes.

TEST PROCEDURE

A large number of tests were made in order that the influence of the numerous variables which affect the air-entraining properties of a concrete mix could be established with a reasonable degree of certainty. A range of slumps, water-cement ratios, air percentages, brands of cement and types of aggregate were included in the series of 102 concrete mixes. The percentage of entrained air by absolute volume was used as the independent variable throughout the entire investigation, and an effort was made to vary the amount of air in each set of 6 mixes by increments of 1 per cent from approximately 0 to 6 per cent. Agent was added to all mixes to obtain a pre-established percentage of air regardless of the amount of agent required. A standard solution of Vinsol resin was made


SHASTA AGGREGATE
REGRESSION LINE W = 278.91 - 6.19A
CORRELATION COEFFICIENT = 0.92
(0.765) REQUIRED TO BE HIGHLY SIGNIFICANT
MAX. SIZE OF AGGREGATE = 1½"
MIXES WITH AVG. SLUMP OF 3"
MIXES WITH AVERAGE SLUMP OF 1¾"
REGRESSION LINE FOR ALL MIXES WITH SHASTA AGGREGATE
WATER CONTENT - POUNDS PER CUBIC YARD
PERCENT ENTRAINED AIR (BY ABS. VOL.)

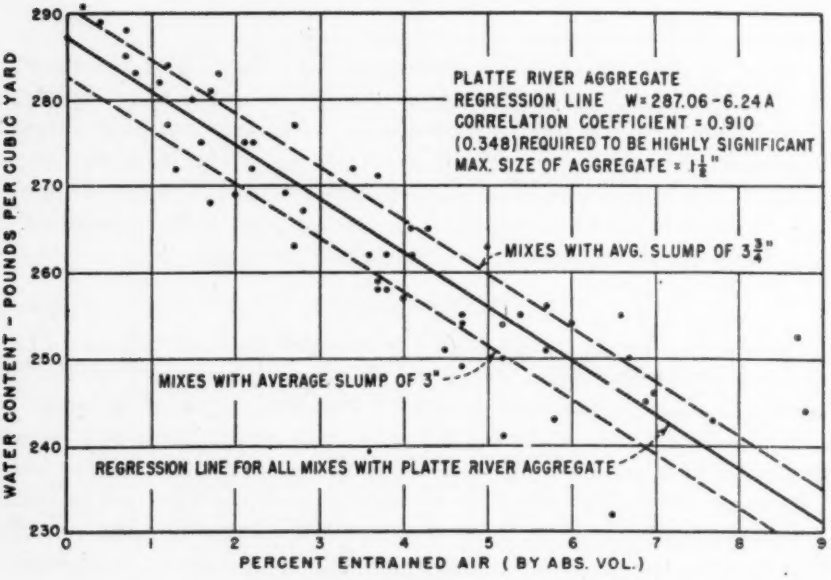
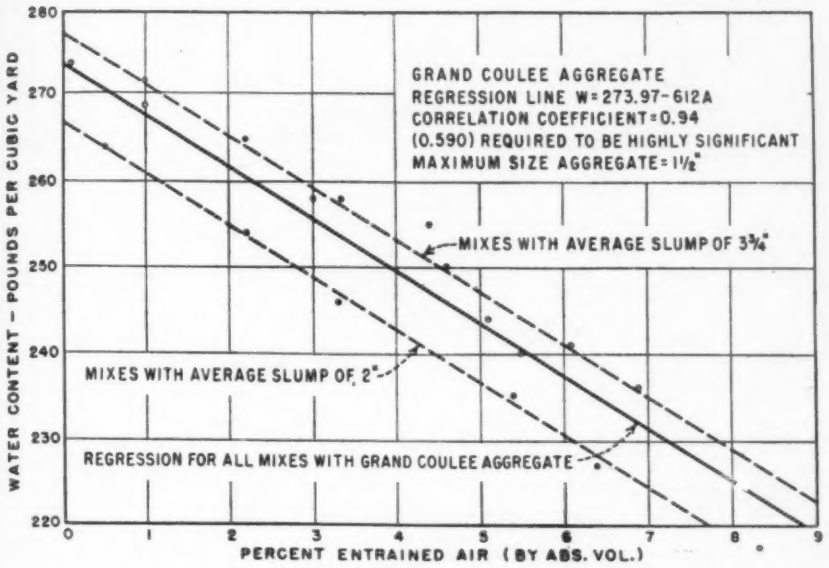
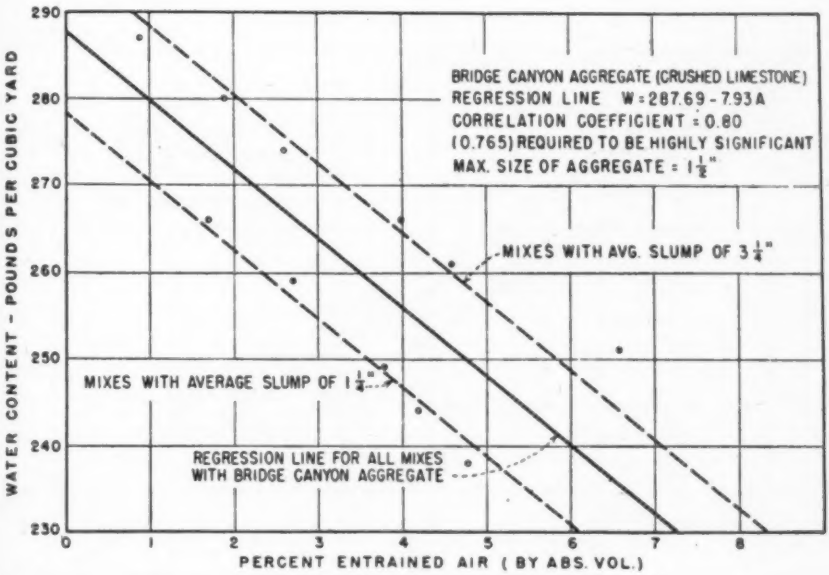

PLATTE RIVER AGGREGATE
REGRESSION LINE W = 287.06 - 6.24A
CORRELATION COEFFICIENT = 0.910
(0.348) REQUIRED TO BE HIGHLY SIGNIFICANT
MAX. SIZE OF AGGREGATE = 1½"
MIXES WITH AVG. SLUMP OF 3¾"
MIXES WITH AVERAGE SLUMP OF 3"
REGRESSION LINE FOR ALL MIXES WITH PLATTE RIVER AGGREGATE
WATER CONTENT - POUNDS PER CUBIC YARD
PERCENT ENTRAINED AIR (BY ABS. VOL.)

Fig. 2a—The decrease in water content due to air entrainment


GRAND COULEE AGGREGATE
REGRESSION LINE W=273.97-612A
CORRELATION COEFFICIENT=0.94
(0.590) REQUIRED TO BE HIGHLY SIGNIFICANT
MAXIMUM SIZE AGGREGATE=1½"
MIXES WITH AVERAGE SLUMP OF 3¾"
MIXES WITH AVERAGE SLUMP OF 2"
REGRESSION FOR ALL MIXES WITH GRAND COULEE AGGREGATE
WATER CONTENT - POUNDS PER CUBIC YARD
PERCENT ENTRAINED AIR (BY ABS. VOL.)


BRIDGE CANYON AGGREGATE (CRUSHED LIMESTONE)
REGRESSION LINE W=287.69-7.93A
CORRELATION COEFFICIENT = 0.80
(0.765) REQUIRED TO BE HIGHLY SIGNIFICANT
MAX. SIZE OF AGGREGATE = 1½"
MIXES WITH AVG. SLUMP OF 3¼"
MIXES WITH AVERAGE SLUMP OF 1½"
REGRESSION LINE FOR ALL MIXES
WITH BRIDGE CANYON AGGREGATE
WATER CONTENT - POUNDS PER CUBIC YARD
PERCENT ENTRAINED AIR (BY ABS. VOL.)

is approximately the same for different slump

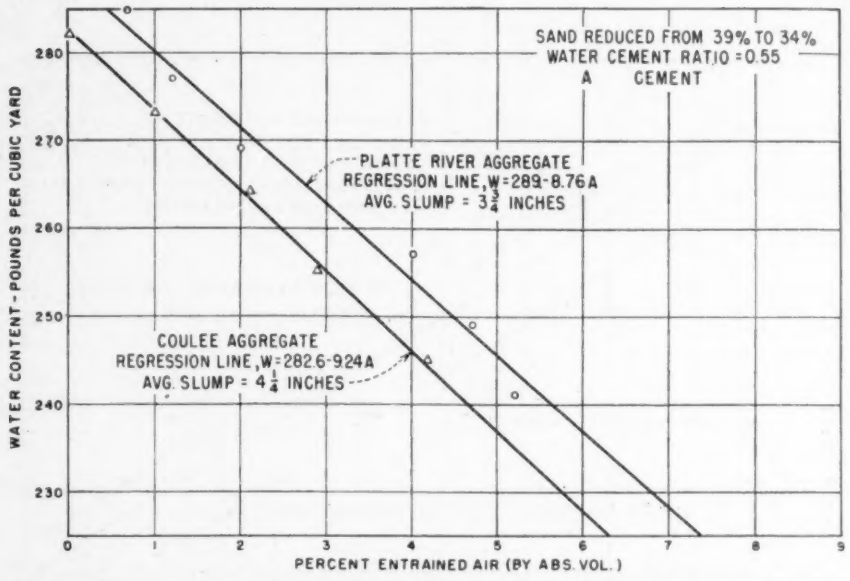

SAND REDUCED FROM 39% TO 34%
WATER CEMENT RATIO = 0.55
A CEMENT
PLATTE RIVER AGGREGATE
REGRESSION LINE, W = 289 - 8.76A
AVG. SLUMP = 3 3/4 INCHES
COULEE AGGREGATE
REGRESSION LINE, W = 282.6 - 9.24A
AVG. SLUMP = 4 1/4 INCHES
WATER CONTENT - POUNDS PER CUBIC YARD
PERCENT ENTRAINED AIR (BY ABS. VOL.)

Fig. 2b—If the percentage of sand is reduced by one for each per cent increase in air content, a reduction of 9.0 lb. of water per cu. yd. is realized for each per cent of air entrained

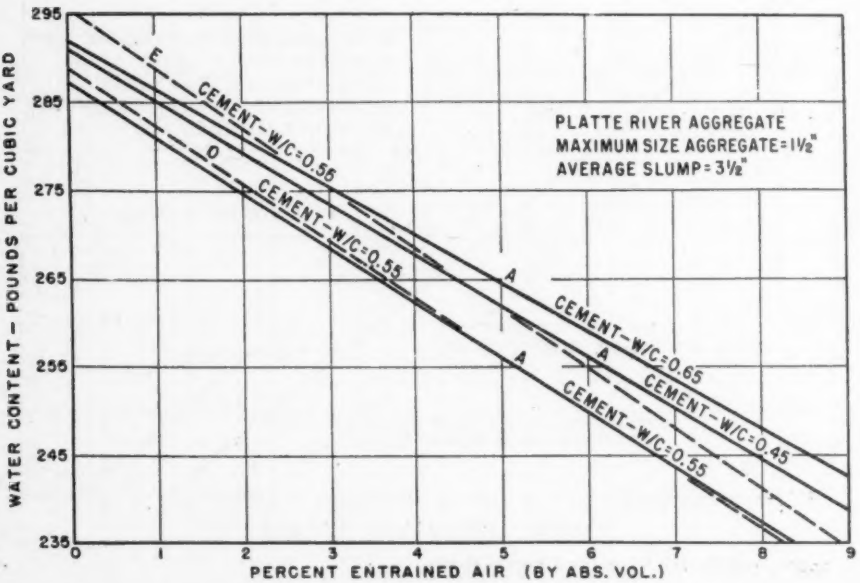
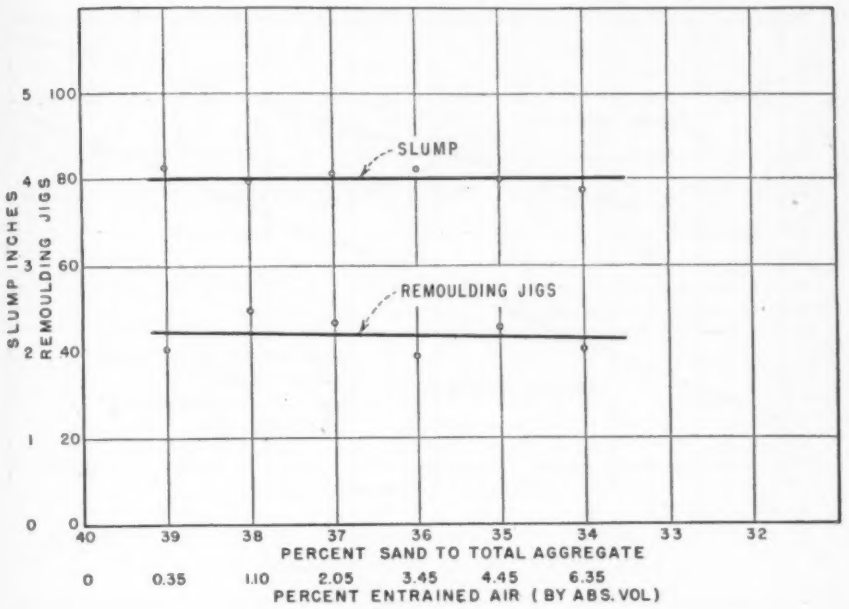


Fig. 3—The reduction in water content for each increase in air entrained is about the same regardless of the water-cement ratio or brand of cement



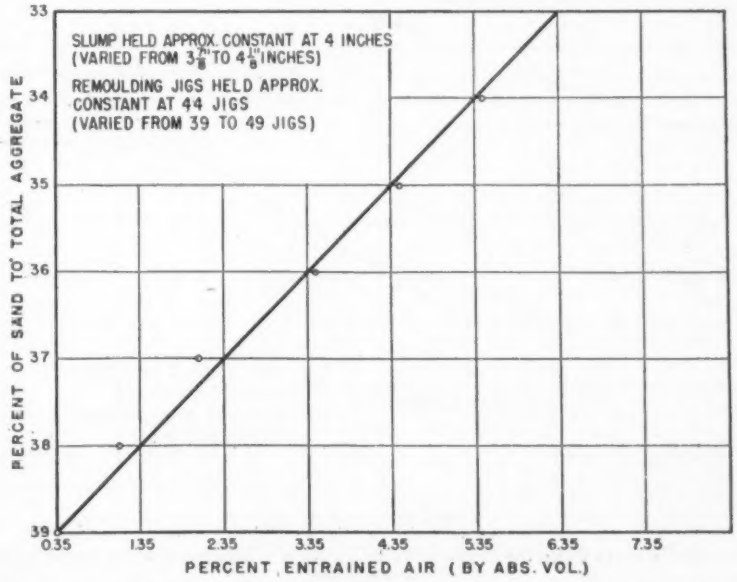

SLUMP HELD APPROX. CONSTANT AT 4 INCHES
(VARIED FROM 3 7/8 TO 4 1/8 INCHES)
REMOULDING JIGS HELD APPROX.
CONSTANT AT 44 JIGS
(VARIED FROM 39 TO 49 JIGS)
PERCENT OF SAND TO TOTAL AGGREGATE
PERCENT ENTRAINED AIR (BY ABS. VOL.)

Fig. 4—The workability and slump remain about the same if the percentage of sand is reduced by one for each per cent increase in air content

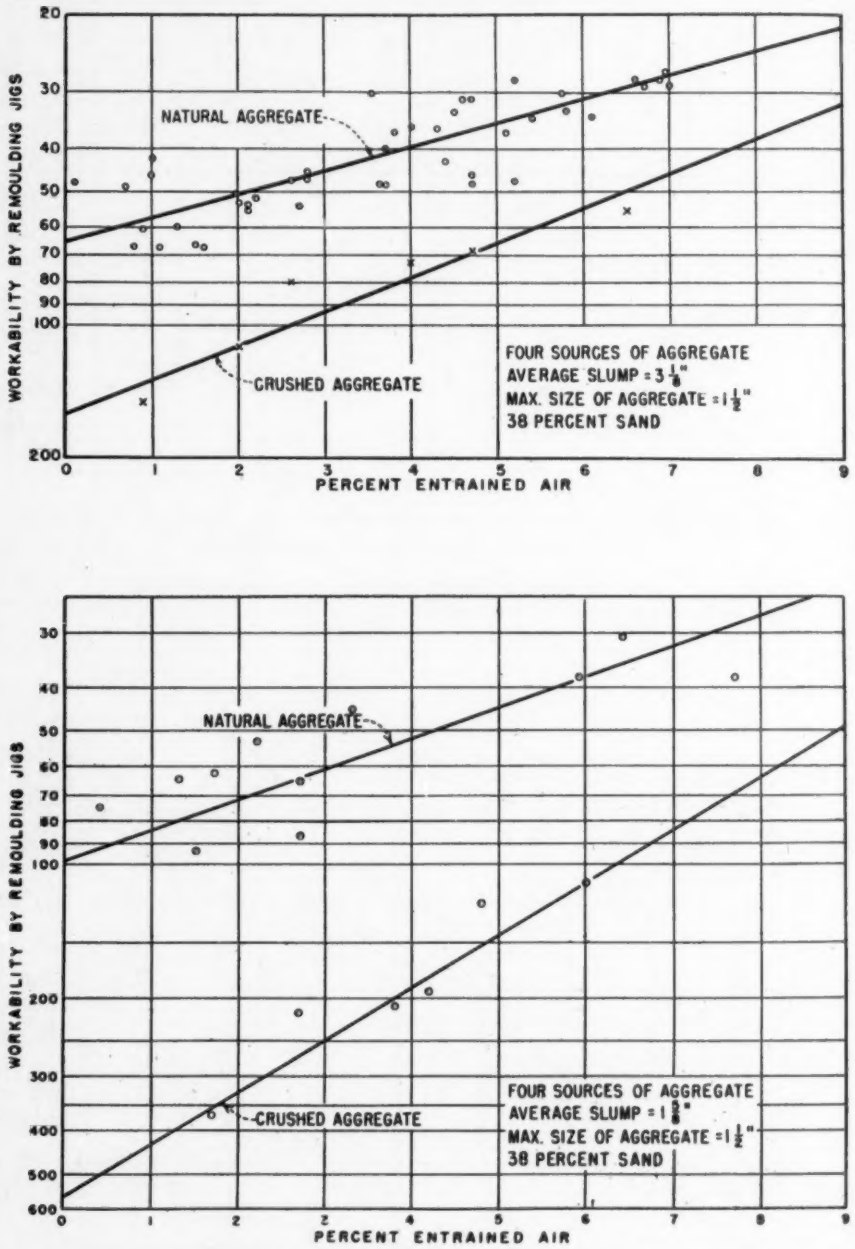


Fig. 5—At the same slump the workability of concrete is increased as the percentage of air is increased

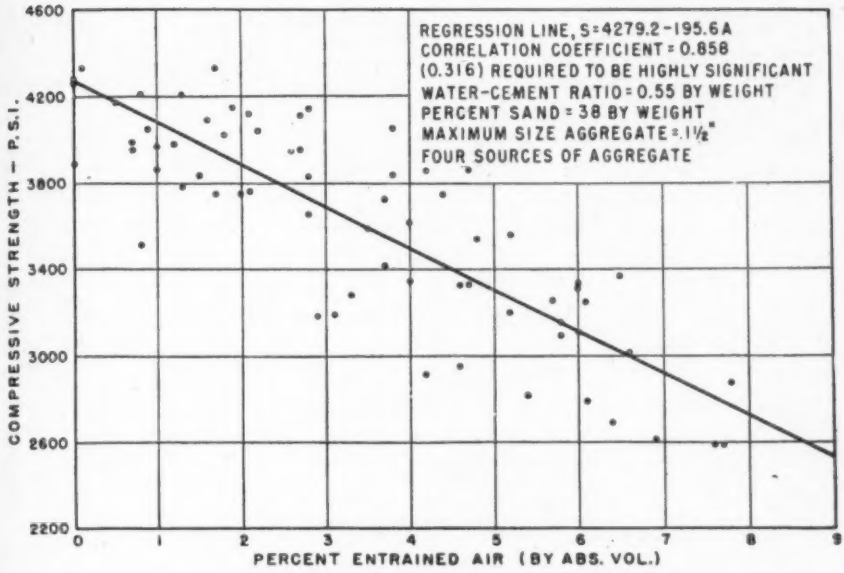


Fig. 6—Concrete having the same water-cement ratio and the same aggregate grading is reduced in strength approximately 195 psi for each percentage increase in air content

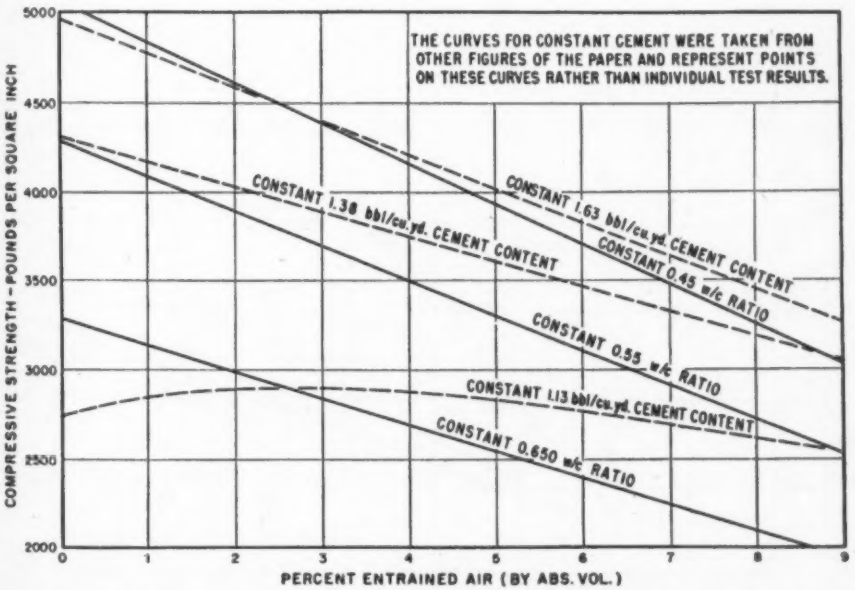


Fig. 6a—The decrease in strength as the air content increases is about the same for different water-cement ratios but for constant cement contents is influenced by the amount of cement in the mix

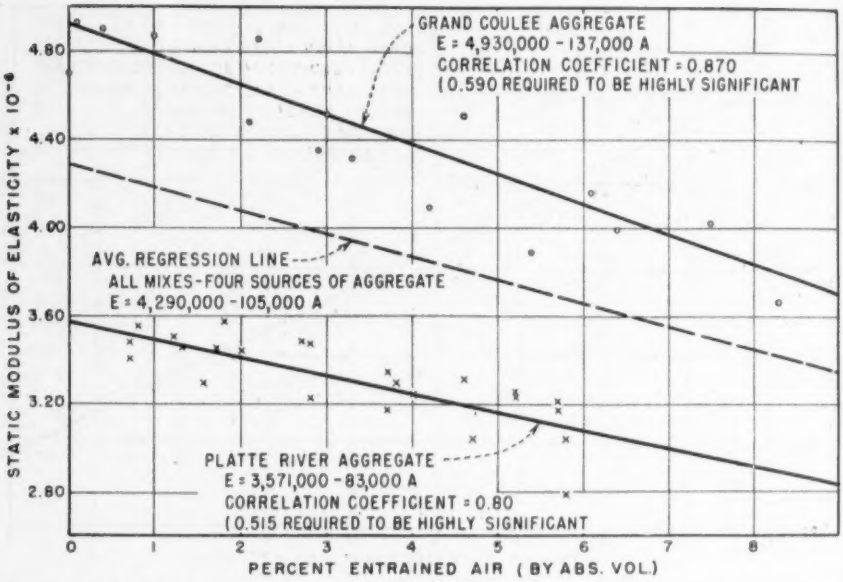


Fig. 7—Modulus of elasticity is decreased 105,000 psi for each percentage increase in air content

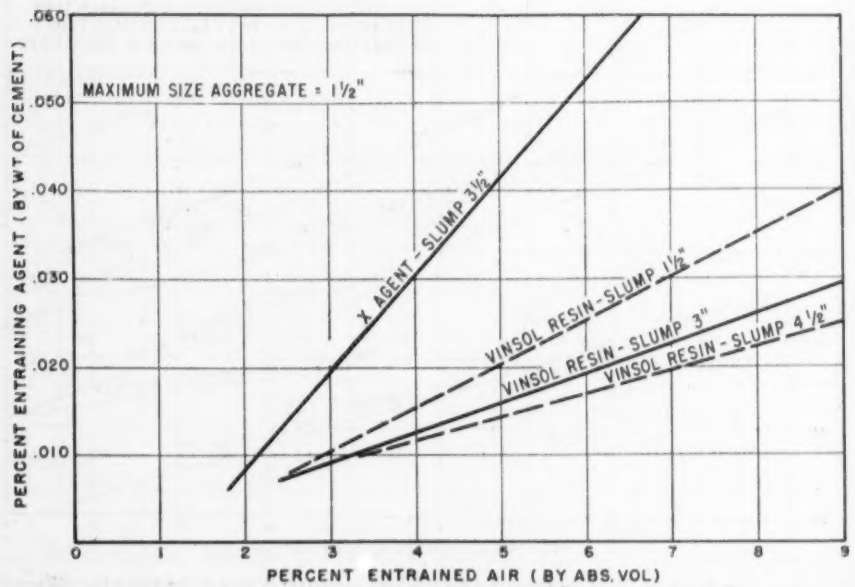


Fig. 8—The amount of air entrained in a given mix, is influenced by the slump and the kind of agent used

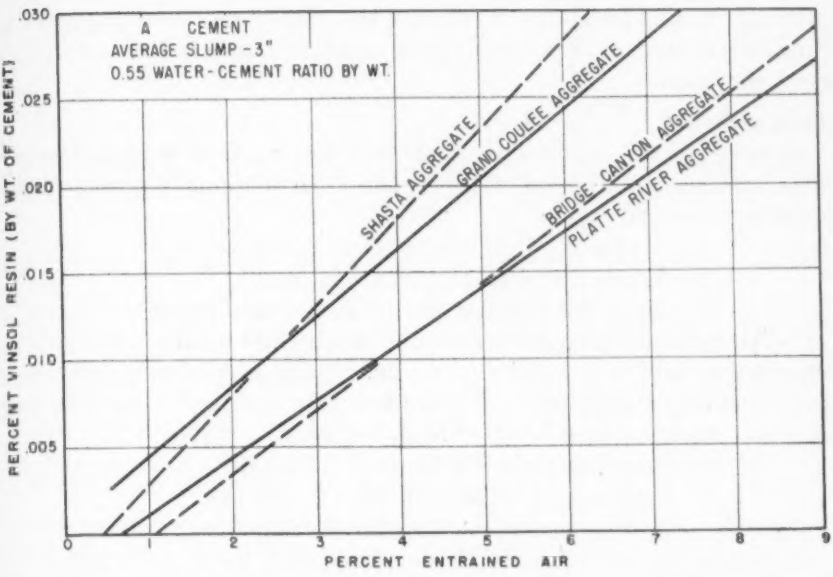


Fig. 9—The amount of air entrained in a given mix, is influenced by the aggregate used

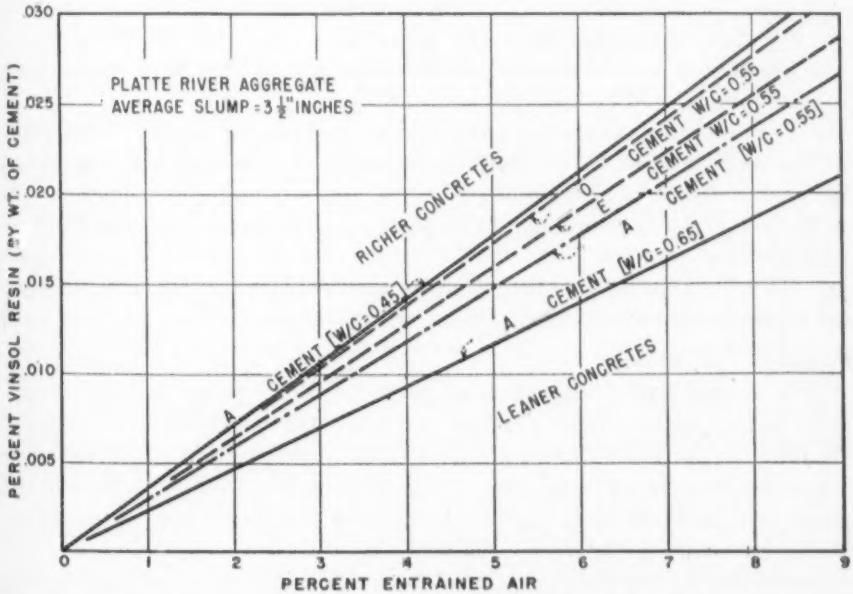


Fig. 10—The increase of entrained air in a given mix is influenced by the brand of cement and the water-cement ratio

before the mixing program started, and this supply was used throughout, with the exception of 12 mixes in which another commercial air-entraining agent was used.

Mixing procedure

All mixes of this study were made in a 1½ cu. ft. tilting laboratory mixer with a mixing period of 5 minutes. The charging sequence of the mixer was as follows:

1. One half of the mixing water and agent
2. Sand, gravel and cement combined
3. Remaining half of mixing water and agent.

The air-entraining agent was combined with the mixing water before introduction into the mixer. After the mixing period, the mixer was discharged into a large pan and re-worked with a shovel. The following tests were made on each batch of fresh concrete:

Slump test—A.S.T.M. Des. (C143-39)
Unit weight—A.S.T.M. Des. (C138-44)
Powers Remolding Test for Workability[4].

At 28 days three 6 x 12 in. cylinders from each batch were tested for compressive strength, A.S.T.M. Des. (C192-44T), and elastic properties.

ADJUSTMENTS AND COMPUTATIONS

Table 5 of the ACI Standard Recommended Practice for the Design of Concrete Mixes[2] could be slightly modified for air entrained concrete by including the following information:

(a) For concrete containing entrained air reduce the sand percentage by one for each per cent of air entrained.

(b) For concrete containing entrained air reduce the water content 8.5 lb. for well rounded natural aggregate and 10.5 lb. for angular aggregate for each per cent of air entrained.

The following adjustment can be placed as a footnote to Table 2 of the standards:

(a) For the same W/C ratio reduce strengths shown in table by 200 psi for each per cent of air entrained in concrete.

Computation of trial mixes

It is very simple to slightly modify the trial-mix computations as outlined in the recommended practice and allow for entrained air by simply treating the air as another ingredient of the concrete mix which replaces an equal volume of aggregate. The sample computations shown on pages 657 and 658 of the above mentioned standards can be modified for air-entrained concrete as follows:

(Assume 3.0 per cent air).

$$\text{Cement content} = \frac{\text{net water content}}{\text{water-cement ratio}}$$

$$= \frac{305}{0.53} = 575 \text{ lb. per cu. yd.}$$

$$= \frac{575}{94} = 6.12 \text{ sacks per cu. yd.}$$

Absolute volume water + cement

$$= \frac{\text{water content}}{62.4} + \frac{\text{cement content}}{\text{specific gravity} \times 62.4}$$

$$= \frac{305}{62.4} + \frac{575}{3.15 \times 62.4} = 7.81 \text{ cu. ft. per cu. yd. concrete}$$

Absolute volume air = per cent air × cu. ft. per cu. yd. of concrete
= 0.03 × 27 = 0.81 cu. ft. per cu. yd. of concrete

Absolute volume total aggregate
= 27 − absolute volume (water + cement) − absolute volume air
= 27 − 7.81 − 0.81 = 18.38 cu. ft. per cu. yd. of concrete

Absolute volume sand = per cent sand × absolute volume total aggreate =
0.42 × 18.38 = 7.72 cu. ft. per cu. yd. of concrete

Absolute volume coarse aggregate
= absolute volume total aggregate − absolute volume sand
= 18.38 − 7.72 = 10.66 cu. ft. per cu. yd. of concrete

Sand content = absolute volume × specific gravity × 62.4
= 7.72 × 2.65 × 62.4 = 1,277 lb. per cu. yd. of concrete

Coarse aggregate content
= 10.66 × 2.55 × 62.4 = 1,696 lb. per cu. yd. of concrete

$$\text{Trial-mix proportions} = \frac{575}{575} : \frac{1,277}{575} : \frac{1,696}{575}$$

$$= 1 : 2.22 : 2.95, \text{ say } 1 : 2.2 : 3.0$$

Computation of air content

The computation of the air content of freshly mixed concrete naturally becomes an important function in the design of concrete mixes in which air-entraining agents are used. A.S.T.M. Des. (C173-42T) "Test for Air Content of Concrete" is not recommended for mortar containing soap forming materials and is not widely used for air-entrained concrete. The most popular method of determining the percentage of entrained air is A.S.T.M. Des. (C138-44) which assumes that the difference between the unit weight computed from the absolute volume of the cement plus water plus aggregate and the unit weight determined with the freshly mixed concrete is due to the amount of entrained air in the mix. The percentage of entrained air is therefore equal to

$$\text{Per cent air} = \frac{(\text{Theoretical unit weight} - \text{measured unit weight}) \times 100}{\text{theoretical unit weight}}$$

Using weights computed in the foregoing trial-mix computation, the per cent air can easily be determined as follows: (Measured unit weight = 142.7 lb. per cu. ft.)

$$\text{Theoretical unit weight} = \frac{\text{weight cement} + \text{weight water} + \text{weight aggregate}}{\text{solid volume of cement} + \text{water} + \text{aggregate}}$$

$$= \frac{575 + 305 + 2{,}973}{27 - 0.81}$$

$$= \frac{3{,}853}{26.19} = 147.1 \text{ lb. per cu. ft.}$$

and

$$\text{Per cent air} = \frac{147.1 - 142.7}{147.1} \times 100 = 3.0 \text{ per cent}$$

Designation 24 of the U. S. Bureau of Reclamation Concrete Manual outlines a general method for the computation of per cent voids (per cent air) based on the actual mix parts of any combination of cement, water and aggregate. A simplification of this method is presented in the following general formula:

$$\text{Per cent air} = 100 - \frac{\left(\frac{1}{G_C} + W/C + \frac{P_A}{G_A}\right)100\,W}{62.4\,P_T}$$

where

P_T = Total parts (cement, aggregate, water) in mix (by weight)
P_A = Parts of aggregate (by weight)
G_C = Specific gravity of cement
G_A = Specific gravity of aggregate
W = Measured unit weight of fresh concrete (lb. per cu. ft.)
W/C = Water cement ratio by weight.

ACKNOWLEDGMENT

The tests were made in the Denver laboratories of the U. S. Bureau of Reclamation under the direction of Robert F. Blanks, Chief, Engineering and Geological Control and Research Division and were supervised by W. H. Price, Head of the Materials Laboratories. The writer is indebted to Mr. Price and to L. H. Tuthill for many valuable suggestions and criticisms throughout this investigation and to the laboratory personnel for valuable assistance in conducting the tests.

All design, construction, and research work of the Bureau are under the supervision of Walker R. Young, Chief Engineer; and all activities of the Bureau are under the direction of Michael W. Straus, Commissioner.

REFERENCES

1—George W. Snedicor, "Statistical Methods."

2—ACI Standard Recommended Practice for the Design of Concrete Mixes. ACI *Proceedings*, Vol. 41, June 1945, page 651.

3—Symposium: Concretes containing air-entraining agents. ACI *Proceedings*, Vol. 40, June 1944, page 509.

4—T. C. Powers, ACI *Proceedings*, Vol. 28, February 1932, page 419.

5—Charles E. Wuerpel, ACI *Proceedings*, Vol. 42, February 1946, (Bibliography page 355).

6—A.S.T.M. Standards 1944.

To facilitate selective distribution, **separate prints** of this title (42-26) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946.

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Recent Experiences with Air-Entraining Portland Cement Concrete in the Northeastern States*

By L. E. ANDREWS†

SYNOPSIS

Describes preliminary studies and current practice, in the northeastern states in the reduction of frost action on concrete paving and other projects (roads, streets, airport pavements and hangars, bridges and buildings); mix design, methods of determining air content and field control. Specifications tend to limit air content to 3 to 6 per cent, rather than fix amount of air-entraining agent used.

The first activity with air-entraining concrete in the northeastern states area was started in 1938. The next 4 years, up to the war period, embraced the construction of experimental projects by numerous states and other agencies. For example, the highway departments of New York, Pennsylvania, Massachusetts, Maine and Vermont used air-entraining portland cement, in addition to other cement variables, for comparative performance purposes in these experimental jobs. The results with A-E concrete proved to be highly satisfactory.

New York was the first state in this group to start such experimental work. Most of these projects were built in 1939-40. Last year I had occasion to make close observations on this work by walking the entire length of these jobs and noting all details of existing condition. What we believe to be more complete information today, in the light of past experiences, was naturally not known or available at the time these pavements were built. Nevertheless, all of the sections observed on this inspection were in excellent condition showing no scaling or disintegration of any kind. However, in many instances, considerable scaling was observed on adjacent sections where A-E portland cement was not used.

In fact, our observation is that where it is known that at least 3 per cent air entrainment was secured in the fresh concrete, and normal

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Regional Highway Engineer, Portland Cement Association, New York City.

materials and construction methods used, there has been no subsequent scaling or other disintegration.

During the war period great impetus was given to the use of A-E portland cement concrete in military installations and particularly at airports for paving and, in fact, also for much structural work. The War Department became one of the largest users and extensive studies were made by the Central Concrete Laboratory of the Corps of Engineers.

Since V-J day and the advancing of postwar programs, particularly in the paving field, attention has been focused on the desirability of benefitting from this previous experience with A-E concrete by providing for its use on current and future work by definite specification adoption.

At this time most of the northeastern states have either adopted specifications for A-E concrete for paving work or are in the process of preparing such specifications.

During the early use of A-E portland cement concrete in this area there were a few instances where too much air was developed in the mix. This was under the old specification where fixed limits were placed upon the amount of A-E agent interground with the cement.

It was soon recognized that the same percentage of A-E agent in different cements would not necessarily induce similar amounts of air in the same class of concrete mixture. It then became more than ever apparent that the important feature is control of the air content and this could not be insured by rigidly fixing the limits for the amount of A-E agent used in the manufacture of the cement.

CURRENT PRACTICE

Now we have the improved specification represented by A.S.T.M.: C175-46T which provides for a permissible range of 12 to 20 per cent air entrainment in a standard mortar test.

It is expected that this latest revision will preclude any tendency toward less than minimum air content in the fresh concrete. It therefore appears that the possibility of securing occasional mix combinations resulting in air content outside the desired range of 3 to 6 per cent has been narrowed considerably.

The principal field of use for A-E concrete in the northeast has been that of pavements for airports, roads and streets. It has also been introduced into many other types of structures including hangars, buildings, bridges, piling, flood control facilities, and other miscellaneous uses. The U. S. Engineer Office in this area, reported last year that A-E portland cement concrete had been used on 170 projects of all kinds in their various activities.

More than a dozen major air bases in which A-E portland cement concrete was used for the paving, embracing considerably more than 3,000,000 sq. yd., were constructed here during the war years 1942-45. One of the largest of these was Stewart Field, West Point, with a total of 1,300,000 sq. yd.

Numerous A-E concrete paving projects have also been constructed during these years in Pennsylvania, New Jersey, New York, Connecticut, Massachusetts, Vermont and Maine, and practically without exception they show no scaling or disintegration.

CONCRETE MIX DESIGN

Concrete mix design and control on all of this work has been simple. Generally, the mix set-up has been based on proportions ordinarily used with normal portland cement for similar work but with sand and water content reduced by an amount to compensate for volume occupied by the increased air-content thus maintaining constant yield.

Such adjustments allow for the same cement factor and about the same consistency. Workability of the resulting concrete is much improved, segregation is minimized and bleeding is considerably reduced. This latter quality allows finishing operations on concrete paving to keep close behind the paver.

On paving work where gravel coarse aggregate is used, water ratios have ranged from about 4.75 to 5.25 gal. per bag of cement with sand-aggregate ratios (by absolute volume) from 29 to about 34 per cent. Where crushed stone coarse aggregate is used the water ratio range has been from about 5.00 to 5.50 gal. per bag while sand-aggregate ratios have been about 33 to 38 per cent. The slump has ranged from about 1½ to 2½ in.

DETERMINING AIR CONTENT

To date, air entrainment has generally been determined by the gravimetric method using the standard unit weight test, comparing actual unit weight of the concrete with calculated air-free unit weight. This method requires accurate data on specific gravities, free moisture, absorbed moisture, water added at paver, and batch weights.

In some instances where normal portland cement has been available on the job at the start of paving operations, it has been used in establishing a unit weight of concrete for basis of comparison with the unit weights secured later with A-E portland cement. In such cases it has been assumed that the air content for the normal cement mix is 1 per cent. Usually, however, where A-E portland cement is specified there is no other cement on the job and the calculated air-free unit weight must then be determined.

Considerable interest has been expressed relative to the "Indiana Method" of determining air content by use of the hook gage in combination with the usual equipment. If close agreement is shown with the standard gravimetric field method in determining volume of air, it would appear that this method may receive wide use because it affords a direct measurement.

FIELD CONTROL

It is always of particular interest to know what was done to correct conditions resulting in air content falling outside the desired range. These occurrences have been few in number, and, as noted above, it is believed that current specification changes will tend to prevent repetition of such instances. However, on a railroad job last year it was necessary to add a neutralized Vinsol resin solution to the batch in order to increase the air content from less than 2 per cent to slightly above 3 per cent. The quantity added was calculated to be about 0.002 per cent which brought the total in this case to about 0.034 per cent.

In another instance, on a highway job, it was possible to increase the air content about 1 per cent by increasing the sand-aggregate ratio (gravel coarse aggregate) from 29 per cent to about 36 per cent.

On a prewar airport project where excessively high air content occurred, it was necessary to blend off with normal portland cement in order to reduce the air to desired limits. Such instances have been the exception and not the rule.

INCREASED DURABILITY

The use of air-entraining concrete in the northeastern states has resulted in greatly increased durability, that is, resistance to freezing and thawing and salt action in all cases where sufficient air-entrainment was obtained.

Practically all specifications now require that the entrained air in the fresh concrete, as determined by the standard yield test, shall come within the range of 3 to 6 per cent. This requirement, with the selection of proper materials, careful mix design and conscientious field control gives assurance, based on experience over the past 8 years, that scale free concrete pavements are an accomplished fact in our northern states.

To facilitate selective distribution, **separate prints** of this title (42-27) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946.

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Experiences with Air-Entraining Cement in Central-Mixed Concrete*

By ALEXANDER FOSTER, Jr.†
Member American Concrete Institute

SYNOPSIS

Reports studies by Warner Co. with central-mixed concrete using air-entraining cement over a 2½-year period. Following experimental work, more than 30,000 cu. yd. of air-entraining concrete, chiefly of low-slump, plastic mix were used for pavements and other highway work. Studies included effect of truck-mixer or agitator action, closed drum and open top equipment, on hauls up to 45 minutes. No significant differences in slump or air entrainment (2½ to 3 per cent) was found. The chief problem is in added storage requirements to meet specifications permitting no admixtures and those demanding air entrainment.

Warner Company has done a considerable amount of work with air-entraining cement over the past 2½ years. Prior to committing the Company to furnish concrete containing entrained air commercially, experimental work was carried out to familiarize ourselves with its peculiarities. Subsequently, over 30,000 cu. yd. have been delivered, mostly for concrete pavements and highway structures. Consequently, the major part of our experience has been with relatively low-slump, but plastic concrete—in general, with slumps of less than 3 in.

Our experimental work was inspired by some early experiences in the industry which had suggested that excessive amounts of air would be entrained in the ready-mixed concrete operation, with consequent difficulties of control. It was variously claimed that the slow, folding, mixing action of a truck mixer, or agitator, the prolonged mixing incidental to transportation, and the air-tight characteristics of the horizontal drum type of mixer or agitator caused the entrainment of greatly more air than for the usual job mixer. A series of tests was conducted

*Presented at Air Entrainment Session, 42nd ACI Convention, Feb. 19, 1946.
†Vice President, Warner Co., Phila., Pa.

by the Company's concrete engineer, H. J. Knopel, to develop information on these and other points. The cooperation of representatives of the Pennsylvania Highway Department and Stanton Walker of the National Ready Mixed Concrete Association was had in the outline and conduct of the investigation. The experience of these tests, reported elsewhere*, did not support the claims that pre-mixed concrete entrained more air than that produced by job mixing. The Company accepted contracts to furnish concrete to several large highway jobs, with satisfactory results to all concerned.

The experimental work, conducted in 1944, had shown that the use of some cements would result in the entrainment of too much air. It became clear that mixing time, but *not* the nature of the mixing action or of the mixing drum, tended to increase the amount of air entrained in some cases. Apparently that was due, according to the generally accepted explanation, to the universal use, at that time, of the dry flake material (not preneutralized) for intergrinding with the cement. Whatever the cause of this variable the air-entraining properties of the cement were investigated in concrete in the laboratory before being accepted for commercial work. This was in addition to the usual test to determine conformity with A.S.T.M. specifications. That practice resulted in the rejection of some cement but avoided complications on the job. As a further precaution weight per cu. ft. tests of the concrete from which air content could be computed, were made throughout the progress of all work. Even today, before cement is accepted for a large road job, samples of the specific brand to be used are obtained and tests are run in the laboratory with the actual amount of cement, water and particular quantities and grading of fine and coarse aggregate to determine the air entrainment by means of the wet weight of the resulting concrete.

With the adoption of limitations on the amount of air entrained in a standard mortar test (A.S.T.M. Method C 185), and perhaps with changes in technic of cement manufacture, no difficulties have been experienced with excessive air entrainment. On the contrary, the tendency has been in the direction of entraining an amount of air near or slightly below the generally accepted minimum.

The need for paving several areas in one of the Company's distributing yards, afforded an opportunity to conduct experiments with full-sized batches of concrete. Centrally mixed concrete, in 5¼ cu. yd. batches, was transported for various periods up to 90 minutes in both a horizontal drum and a high-discharge type of truck agitator. For another paving job we used an open-top dump body. Concrete, in 3 cu. yd. batches, was transported for periods up to 45 minutes.

*A.S.T.M. *Bulletin*, October, 1944; and ACI JOURNAL, June, 1944, *Proceedings*, V. 40, p. 547.

The truck agitators were revolved at the slow speed of 2 r.p.m.—our standard practice. For the periods investigated, no significant effect on either the amount of air entrained or the slump was found. With normal atmospheric temperatures the recession in slump seems to be less with air-entraining cement. However, in hot weather, or when heated concrete is provided in cold weather, recession in slump for air-entraining-cement concrete requires the usual compensating amounts of water. It should be pointed out, however, that although an air-entraining cement was used, an amount of air between only about 2½ and 3 per cent was entrained. A further observation was that no important differences were shown as between the two types of drums. It did appear that these very stiff mixes were discharged somewhat more readily from the horizontal drum than from the high-discharge drum, but the difference was not enough to be considered of practical importance.

The question of the use of non-agitating equipment (open-top dump bodies) has come very much to the fore with the advent of air entrainment. Our work was done with a water-tight body having a sloping rear and no tail-gate. It might be described roughly as being something of the nature of a bath tub. The results with this method of transportation were not encouraging. It is true that the entrainment of air reduced segregation and packing of the concrete, but it did not eliminate them. The top, middle, and bottom strata of the concrete had a decidedly different appearance. Difficulty was experienced in discharging the load from the body. Perhaps more satisfactory results would have been obtained with a better designed container, but our experience suggests that, except for special cases, the need for agitation during transportation is not eliminated by air entrainment.

The experimental field concrete just referred to included concrete made with normal cement and concrete of a wide range in consistency and cement content. Some of the loads with the open-top dump body were made to contain excessive air, by use of sodium hydroxide solution of Vinsol resin as an admixture. Standard cylinders were molded for compression tests at 7 and 28 days. Opportunity has not been had to analyze these data thoroughly and, further, they would be too voluminous to report here.

However, something can be said concerning the effect of air entrainment on strength, based on our routine tests. Mr. Knopel not only designs and controls our concrete mixes, but he makes regular tests of representatives batches of the different classes of concrete each day. A comparison of the average level of strengths for concrete made with air-entraining cements as compared with that for standard cement indicates, for the pavement type of mix, about 80 per cent of normal. These

pavement mixes tend to be under-sanded under usual circumstances and, as air is entrained, we are required to reduce the sand content to compensate for the volume of air. For structural concrete with the somewhat leaner cement and with proportionally higher sand and water content the level of concrete strength for air-entraining cements averages about 90 per cent of the normal cement concrete strength.

Questions have been raised concerning difficulties in handling air-entraining cement in bulk. We have taken pains to observe if there is an unusual tendency of the cement to leak from openings in bins, gates, chutes, screws, etc. No handling difficulties have been encountered which do not appear to be common to all cements.

Questions have also been raised concerning the "dischargeability" of concrete made with air-entraining cements as compared with standard cements. Because of some difficulties which we thought we encountered (they turned out to be for another reason than air entrainment) we paid particular attention to this problem and we also made a survey of experiences over a fair cross-section of the industry. The consensus of our own experience and of the survey is that such differences as exist are negligible and, if anything, favor the concrete containing entrained air. With the air-entraining cement, the concrete seems to have less tendency to cling to the drum surfaces than normal-cement concrete—and, in general, the problem of keeping mixers clean may be somewhat less.

The introduction of air-entraining cements has presented one outstanding difficulty to the ready-mixed concrete producer. It lies in facilities for storage. Under normal conditions, minimum storage facilities for two or three cements are required—Type I or II, and sometimes both, and the high-early-strength Type III. That condition commonly is still further aggravated by the necessity of maintaining special bins for particular jobs. The additional complication presented by the addition of Types Ia and IIa, with Type IIIa in the wind, will be readily recognized. An immediate solution would be to use air-entraining admixtures, with which we have had some limited and, on the whole, satisfactory experience. However, in our area, admixtures are not permitted by certain specifications.

To sum up, Warner Company's management looked with fear on the use of air-entraining cements. Today that fear is gone, although the recognition of new problems of control exist. While we have suffered some loss in strength due to entrained air, we have, nevertheless, been able to maintain the required strengths. Important difficulties of handling either the cement or the concrete have not been encountered. On the other side, the complications introduced by additional storage requirements are serious.

[To facilitate selective distribution, **separate prints** of this title (42-28) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946.]

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Studies of Concrete Containing Entrained Air*

By STANTON WALKER†
Member American Concrete Institute
and DELMAR L. BLOEM‡

SYNOPSIS

Problems of air entrainment in concrete have been particularly interesting to the ready-mixed concrete industry which has to meet a wide variety of specification requirements. This prompted exploratory studies in the Research Laboratory of the National Ready Mixed Concrete Association. Data are reported on effect of entrained air on compressive strength and mixing water requirements. Other factors considered are: mixing time, grading of aggregate, temperature, ratio of sodium hydroxide to Vinsol resin, comparisons of fresh and hardened concrete, and air content at different depths of concrete.

Entrainment of air in concrete has introduced new problems in the control of concrete quality. These problems have been of particular interest to the ready-mixed concrete industry, whose members are required to produce concrete meeting a wide variety of requirements. To develop information on the effects and control of air entrainment, helpful to the ready-mixed concrete manufacturer, a number of studies have been made in the Research Laboratory of the National Ready Mixed Concrete Association at the University of Maryland. Much of that work has been exploratory in nature to obtain information indicating trends.

The more comprehensive tests have dealt with effects of entrained air on strength and mixing water requirements. Other factors studied have included: time of mixing, grading of aggregates, temperature of concrete, make-up of Vinsol resin solution, relationships between fresh and hardened concrete, and miscellaneous phases of the general problem. This paper presents brief summaries of most of these studies. For some

*Presented at Air Entrainment Session, 42nd ACI Convention, Feb. 19, 1946.

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of them, more detailed reports are available and may be obtained on request to the authors.

CONDITIONS OF TESTS

The data summarized are derived from a number of independent investigations. However, the conditions under which they were made are sufficiently similar that a general statement on procedures should be sufficient.

For the tests reported air-entraining admixtures were used, instead of air-entraining cements, to provide for a wide range in air content. Most of the work was with the flake Vinsol resin; some tests were made with the sodium resinate (N.V.X.) and with Darex. The first two are products of the Hercules Powder Co., and the last a product of the Dewey and Almy Chemical Co. The flake Vinsol resin was used in a sodium hydroxide solution and the sodium resinate in a water solution; the Darex was used as received.

The concrete was made with Type I cement and well graded sand and gravel of 1 in. maximum size. Unless otherwise noted it contained 5.5 sacks of cement per cu. yd. It was designed to produce mixtures of good workability and of a consistency represented by a slump of 3 to 4 inches. It was mixed in 0.5 cu. ft. batches in a small tilting mixer of 1 cu. ft. capacity. Except as otherwise noted the mixing time was 6 minutes, at 16 r.p.m., representing a peripheral speed of 90 ft. per minute. With the entrainment of air, the volume of sand was reduced to maintain, in so far as practicable, the same absolute volume of cement and coarse aggregate.

Where strength tests were involved specimens were cured in a standard moist room until tested. Air contents reported were determined by the gravimetric method—i.e., computed from the absolute volumes of the ingredients of the batch. The weight per cu. ft. measurements were made in a 1/5 cu. ft. measure.

EFFECT OF ENTRAINED AIR ON COMPRESSIVE STRENGTH

Fig. 1 summarizes results for 7 and 28-day compressive strengths of concrete containing 4.5, 5.5, and 6.5 sacks of cement per cu. yd. Vinsol resin was added in amounts of 0, 0.005, 0.010, 0.015, and 0.020 per cent of the weight of cement, resulting in air contents up to about 9 per cent. It will be seen that, for the 5.5 and 6.5 sack concrete, strength was reduced as air was entrained. For the lean 4.5 sack mix the strength was slightly increased for air contents up to about 6 per cent. This beneficial effect on the lean mix, it seems reasonable to suppose, was due to the improvement in workability—in spite of the fact that, according to usual standards, the air-free mix was workable. For 5 per cent of added air, an average desirable amount, the results may be summarized as in Table 1.

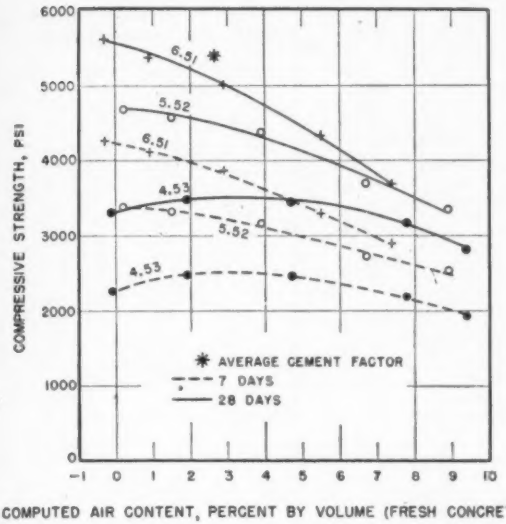


Fig. 1—Relationship between air content and strength (series 66).

TABLE 1

Cement sacks per cu. yd.	Percent change in strength due to 5 per cent added air	
	7-day	28-day
4.5	+9	+4
5.5	−12	−16
6.5	−17	−20

Entrained-air to strength relationships reflect not only the effects of the air but also the reduction in mixing water necessary to maintain the same consistency. The authors, and other writers, have suggested that air affects the strength exactly as so much water. The present data, and other data not reported herein, support the thought that such an assumption affords a good, and generally conservative, basis for design, but that the assumption is not true. Fig. 2 shows 28-day compressive strengths plotted in relationship to the gallons of water *plus* air per sack of cement. Curves, derived by interpolation from the original data, are shown for 0, 2, 4, 6, and 8 per cent of air. A family of curves is revealed, with their slopes decreasing as the percentage of air increases.

Fig. 3 shows the conventional water-cement-ratio-strength relationship for the same data as used in Fig. 2. It may be seen, for example, that concrete containing 4 per cent of air, and having a specified strength, can be obtained by reducing the water-cement ratio about 1 to 1½ gals. per sack below that required to produce the same strength in

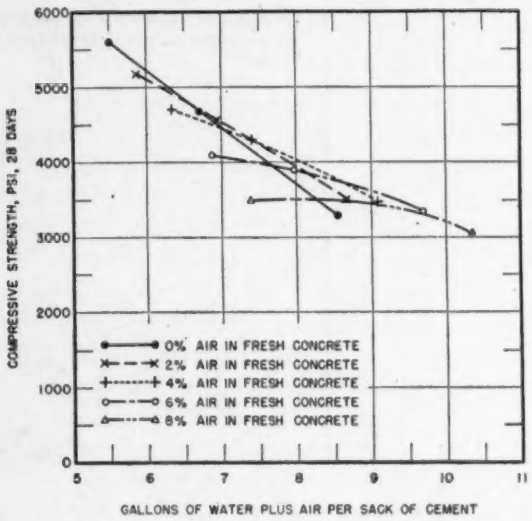


Fig. 2—Strength, water plus air relationships (series 66).

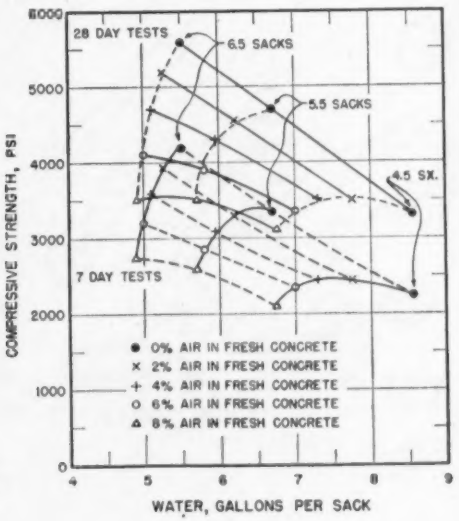


Fig. 3—Relationship between water ratio and strength for various air contents (series 66).

normal cement concrete. Penalties in cement content of from none, for the lower strengths, up to 1 sack per cu. yd., for the higher strengths, are indicated.

EFFECT OF ENTRAINED AIR ON MIXING WATER REQUIREMENTS

As pointed out, requirements for mixing water are reduced as air is entrained. The relation between the volume of air and the reduction in mixing water is so orderly that there can be little doubt but what a

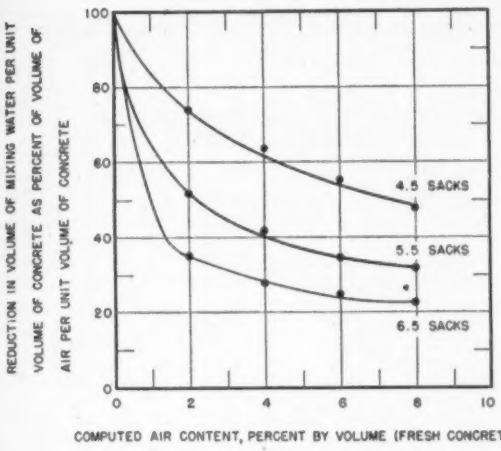


Fig. 4—Effect of entrained air on mixing water (series 66).

rational relationship exists. However, thus far, it has eluded the authors. For the same data as used in earlier diagrams, Fig. 4 shows the amount which the mixing water may be reduced for different cement contents, with the reduction expressed as a percentage of the volume of air entrained in a unit volume of concrete.

The diagram shows that the reduction in mixing water ranged, for the 4.5 sack concrete, from 74 per cent of the air for 2 per cent of air to 48 per cent for 8 per cent of air; for the 5.5 sack concrete the range was from 52 per cent to 32 per cent; and for the 6.5 sack concrete, from 35 to 23 per cent. That is to say, as more air was entrained it became progressively less efficient in reducing mixing water; also, the air was progressively less efficient as the cement content was increased.

EFFECT OF MIXING TIME

In early experiences with air-entraining cements there were some unfortunate cases of the entrainment of excessive air contents, attributed to prolonged mixing. It seems likely that most, if not all, of these cases occurred when using cements containing interground Vinsol resin which had not been pre-neutralized. More recent experiences suggest that, with air-entraining cements as now made, the danger of excessive air contents, due to prolonged mixing as in a ready-mixed concrete operation, is not a serious problem. In fact, some experiences suggest that the air content may be decreased significantly by prolonged mixing.

In view of the absence of quantitative data, tests were carried out with flake Vinsol resin, sodium resinate (N.V.X.), and Darex. The concrete was mixed for periods of 6, 12, 30, and 90 minutes. No significant

differences in results were found for the three air-entraining admixtures and the average results are shown in Table 2.

TABLE 2—EFFECT OF MIXING TIME ON CONCRETE MADE WITH AIR-ENTRAINING ADMIXTURES

Mixing Time	Concrete without Admixture		Concrete with Admixture		
	Compressive Strength	Slump	Compressive Strength	Slump	Air Content
6	98	97	94	93	94
12	100	100	100	100	100
30	98	74	96	74	86
90	98	35	101	35	58

While the data are too few to permit of drawing conclusions some observations on them may be of interest. The results of the slump tests, as well as measurements of air content, suggest that, for this mixer, about 12 minutes were required for adequate mixing. After 12 minutes, both the slump and the air content were decreased by additional mixing. The slumps of both the standard-cement concrete and the air-entraining concrete were affected similarly with mixing time. This result was contrary to expectation, it being reasoned that the reduction in air content would increase the requirements for water and, for that reason alone, the slump would be reduced more when air was entrained than when the concrete was air free.

The data referred to herein, while not conclusive, suggest that, whatever the air-entraining agent, attention to mixing time is required to maintain a uniform air content, when opportunities for wide ranges in mixing time exist. However, after adequate mixing has been done, the relationship is not so critical but what little difficulty should be encountered in exercising the necessary control.

EFFECT OF GRADING OF AGGREGATE

The grading of the aggregate has been shown to affect the amount of air entrained, and this factor was investigated. Principal attention was paid to fine aggregate. Tests of both mortar and concrete were made for sand gradings ranging in fineness modulus from 1.7 to 3.6. These gradings were obtained by adding to relatively fine concrete sand appropriate quantities of a very fine sand (finer than the No. 30 sieve) and small gravel graded between the No. 8 to No. 3 sieves.

A good relationship was found between fineness modulus and amount of air entrained. The quantity of air increased from the lowest fineness

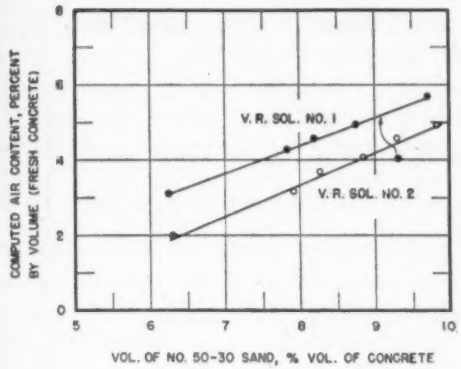


Fig. 5—Effect of grading of sand on air entrainment in concrete (series 63).

modulus to a peak at about 2.5 and, thereafter, decreased sharply. These data are not shown, since other studies suggest that the relationship is not general and that it would differ for other gradings of the same fineness moduli.

The test results were studied along lines suggested by Henry L. Kennedy in his contribution to the 1944 Symposium on Air Entrainment in Concrete*. The proportions of various grain sizes in the sand, expressed as a percentage of the volume of concrete or mortar, were studied in relation to the amount of air entrained. The most satisfactory relationship—or, in fact, the only satisfactory one—was found for air content expressed as a function of the amount of the No. 50 to No. 30 size. The data for concrete, using two different Vinsol resin solutions, are shown in Fig. 5. An equally good relationship, not shown, was found for the mortar.

Only a few tests have been made on the effect of grading of coarse aggregate, but the limited data available indicate that the coarse aggregate has little effect, except in so far as it affects the amount of sand required.

EFFECT OF TEMPERATURE OF CONCRETE

The temperature of the concrete at the time of mixing was found to have a significant effect on the amount of air entrained. By controlling the temperature of the ingredients, concrete was mixed which had temperatures, after mixing, ranging from 46 to 106 F. Vinsol resin, sodium resinate, and Darex were used as admixtures. Results of tests are shown in Fig. 6 and 7.

Fig. 6 shows that, in the case of each admixture, the amount of air entrained decreased as the temperature of the concrete increased. Fig.

*ACI JOURNAL, June, 1944; *Proceedings* V. 40, 1944.

7 is of principal interest. In it, the amount of air is expressed as a percentage of the amount at 70 F. It is interesting that, in spite of the different admixtures and the different amounts of air entrained by the quantities of them used, all points fall on the same curve. At 50 F, the amount of air was 130 per cent and, at 100 F, 77 per cent of that at 70 F.

EFFECT OF RATIO OF SODIUM HYDROXIDE TO VINSOL RESIN

The question has been raised as to how critical is strict adherence to recommended proportions of sodium hydroxide to Vinsol resin in making up the solution for use as an admixture. Tests were made with solutions with the ratio of sodium hydroxide to the resin varied from 0.1, the usual recommendation, to 5.0. The amount of Vinsol resin was maintained at 0.009 per cent of the cement. The results of the data are shown in Fig. 8. It appears that, for the condition of these tests, the activity of the solution increased up to the point where a weight of the sodium hydroxide about equal to that of the Vinsol resin was used. Beyond this point the solution was less active, probably due, as suggested in correspondence with the Hercules Powder Company, to a "salting-out action" caused by the excessive amount of the sodium hyrdoxide.

COMPARISONS OF FRESH AND HARDENED CONCRETE

Interesting relationships between fresh and hardened concrete have been developed. The air content of the hardened concrete was computed from absolute volumes of the ingredients and specific gravity determinations of the concrete. These values were invariably about 20 per cent lower than those for the fresh concrete, also computed from absolute volumes. In search for an explanation, specific gravities of the cement were determined in water. Apparent values were found, ranging from 3.15 after 15 minutes in the water, to 3.35 after 24 hours and to 3.65 after 270 hours, when the tests were discontinued. The use of a specific gravity of cement indicated for 72 hours, about the period at which the displacement of the concrete cylinders was determined, would reduce the differences between the apparent air content of the fresh and hardened concrete to an amount explainable by losses in water in the molding process.

Consideration should be given to the question of whether or not differences in air content indicated by the gravimetric and volumetric methods may not be accounted for, in part at least, by differences in specific gravities of cement determined by conventional methods and by displacement in water.

The differences in the apparent volume of fresh concrete and hardened concrete are an indication, at least, of the shrinkage which takes place on

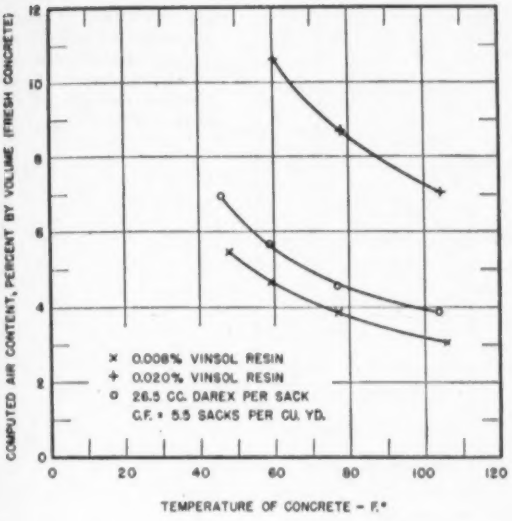


Fig. 6—Effect of temperature on air entrainment in concrete (series 78).

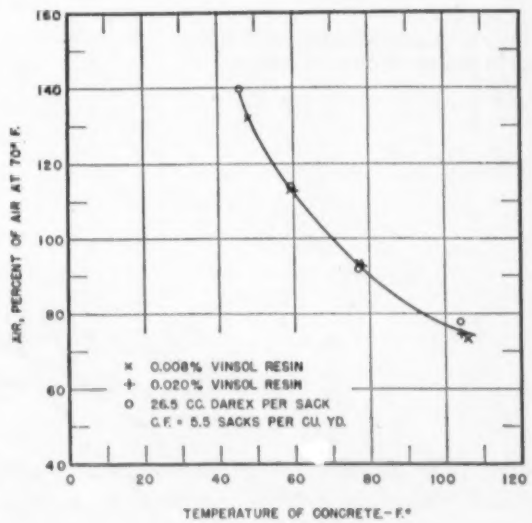


Fig. 7—Ratio of air content at different temperatures to that at 70 F. (series 78).

hardening. These differences increase significantly with increase in air content, as may be seen from Fig. 9. These data are presented to stimulate thinking without attempting to formulate conclusions.

AIR CONTENT AT DIFFERENT DEPTHS OF CONCRETE

Exploratory tests have been made on the effect of depth of section on the air content of the concrete. Columns 6 in. in diameter and 4 ft.

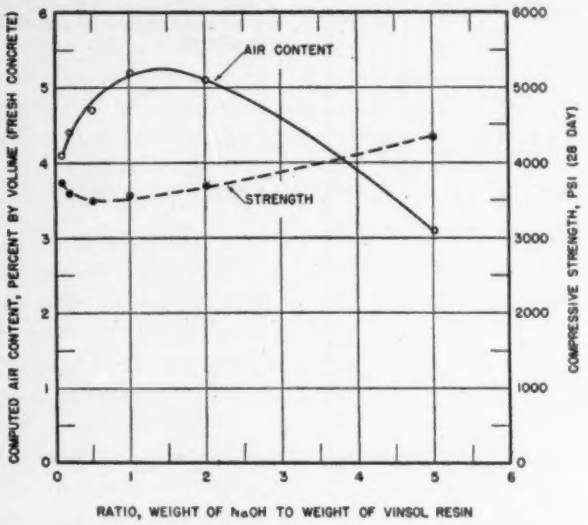
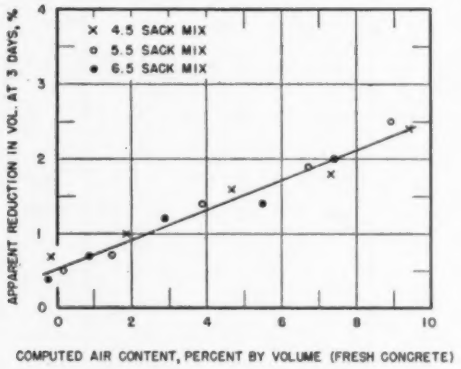

AIR CONTENT
STRENGTH
COMPUTED AIR CONTENT, PERCENT BY VOLUME (FRESH CONCRETE)
COMPRESSIVE STRENGTH, PSI (28 DAY)
RATIO, WEIGHT OF NaOH TO WEIGHT OF VINSOL RESIN

Fig. 8—Effect of amount of sodium hydroxide used to neutralize Vinsol resin on air content and compressive strength of concrete (series 79).

Fig. 9—Apparent reduction in volume of fresh concrete after 3 days (series 66).


4.5 SACK MIX
5.5 SACK MIX
6.5 SACK MIX
APPARENT REDUCTION IN VOL. AT 3 DAYS, %
COMPUTED AIR CONTENT, PERCENT BY VOLUME (FRESH CONCRETE)

high were molded. The concrete was placed in 12 in. "lifts" each of which was rodded 25 to 30 times as required in molding 6 by 12 in. cylinders. After the concrete had hardened, the column was broken into four pieces and the air content of these computed. There was a decrease in the indicated amount of air with distance from the top of the column. However, it appears that these differences may be satisfactorily accounted for by the compression of the air due to the weight of the superimposed concrete. That is to say, the data suggest, contrary to an earlier statement by the senior author, that there is probably no significant movement of air from the bottom to the top of the section for the method of placement used.

CLOSURE

While the tests reported are fairly limited in scope, they do lead to the following observations:

1. Entrainment of air decreased the strength of rich and moderately rich mixes but, in reasonable quantities, increased the strength of lean mixes.

2. Entrainment of air reduced requirements for mixing water. As a substitute for mixing water in maintaining consistency, air was progressively less efficient as the amount of air was increased and as the amount of cement was increased.

3. The amount of entrained air was decreased by prolonged mixing; the slump of air-free concrete and concrete containing entrained air was similarly affected by mixing time.

4. The amount of air entrained was significantly affected by the grading of the fine aggregate and appeared to be principally a direct function of the amount of the No. 50 to No. 30 grain size.

5. More air was entrained by concrete of low temperature than by concrete of high temperature.

6. Increasing the ratio of sodium hydroxide to Vinsol resin up to 1 resulted in increased air entrainment.

7. The differences between apparent volumes of fresh and hardened concrete increased with amount of entrained air.

8. The volume of air at different depths of concrete was affected by the weight of the superimposed concrete.

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Homogeneity of Air-Entraining Concrete*

By HENRY L. KENNEDY†
Member American Concrete Institute

SYNOPSIS

Reports studies of the homogeneity of air-entraining concrete by means of a test for resistance to abrasion. An air-entraining agent was used with no added accelerators, dispersing agents or gas forming materials. "Since air entrainment has a tendency to eliminate bleeding of concrete it is only reasonable to believe that such concretes are more homogeneous . . . . the top surface (of a slab) would have the same strength and density characteristics as the bottom surface." Plotted data show that there is progressive increase in abrasion loss in specimens as air content increases beyond 6 per cent.

Our most recent work in connection with air-entraining agents has involved a study of the homogeneity of air-entraining concrete, utilizing abrasion resistance as a criterion. A common, everyday air-entraining agent was used for these studies—no added accelerators, dispersing agents, or gas-forming materials.

Since air entrainment has a tendency to eliminate bleeding of concrete, it is only reasonable to believe that such concretes are more homogeneous. For example, the top surface would have the same strength and density characteristics as the bottom surface.

Fig. 1 was made in connection with our earlier studies of the homogeneity of high and low bleeding concretes with no air entrainment. Reduced bleeding is accompanied by an increase in homogeneity as measured by compressive strengths.

Tests of air-entraining concretes indicate that there is little difference between the homogeneity of an air-entrained concrete and a relatively high-bleeding, normal concrete, despite the fact that in the case of air-

*Presented at Air Entrainment Session, 42nd ACI Convention, Feb. 19, 1946.
†Dewey & Almy Chemical Co., Cambridge, Mass.

entraining concrete the bleeding is reduced greatly or eliminated altogether. The range in strength and density of many low-bleeding air-entraining concretes has been found to be practically the same as in higher bleeding concretes in the absence of air-entrainment.

A lower density on the upper surface of a concrete pavement would indicate that the surface was more resistant to freezing and thawing, and hence the apparent lack of homogeneity is a favorable factor. In reinforced concrete, however, particularly continuous beams, where the negative reinforcing is near the top surface of the beam, this situation will demand more study. It represents a probable explanation for differences in bond strength between top and bottom of a beam made with air-entraining concrete.

As a means of studying homogeneity, the Dewey & Almy Chemical Co. Laboratories have developed an abrasion test which seems to bring out the differences in relatively shallow slabs, differences which are not readily discernable in strength and density tests.

The apparatus used is a Reumelin sand blast cabinet, except that iron filings are used instead of sand. The air pressure is held constant at 60 psi. The specimen is set 1½ in. away from the nozzle and the abrasion applied for a total of twenty seconds, over an area 1¼ in. by 1½ in. Eight such tests are made on each surface by use of a template.

Fig. 2 shows the effects of abrasion on the top, side and bottom of a cast specimen of an air-entraining concrete containing 10 per cent air.

The figure shows a discernable difference between the resistance to abrasion between the top, side and bottom of the specimen. The actual weight loss in grams, due to abrasion, were as follows: Top 48; Side 33; Bottom 17. On the side view the top of the specimen as cast is to the left. Note the difference in abrasion between the top and bottom of slab even in a 4 in. depth.

Actually, densities were not measured at the top and bottom of this beam. It would have been difficult to discern the difference in density, but the difference in abrasion loss indicates a difference in the concrete from top to bottom as cast.

The abrasion loss noted on the corresponding blank specimen, having a total air content of 0.6 per cent, was: Top 24; Side 13; Bottom 12.

The significant figures are those represented by the abrasion loss on the side of the specimen, as compared with the bottom of the specimen, since both were cast against a steel form. The top of each specimen was simply trowelled smooth. Hence the top of one specimen may be compared with the other, but the homogeneity should be evaluated only on the basis of side and bottom of specimen.

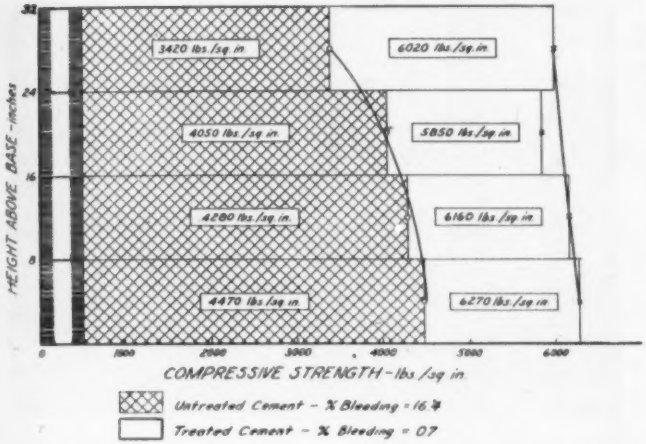

Fig. 1—Air entrainment tends to increase hemogeneity in concrete.

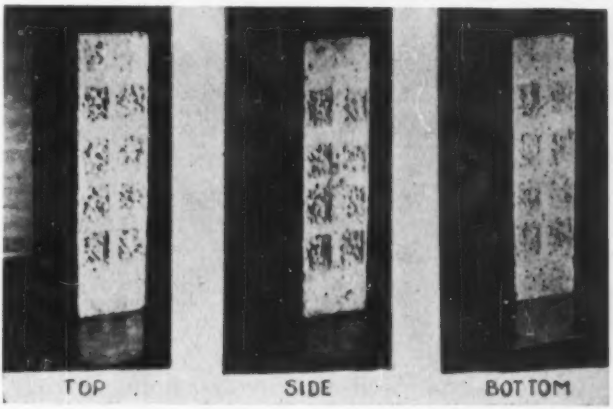

Fig. 2—Abrasion on top, side and bottom of cast specimen — 10 percent air.

It will be noted that in the case of the blank there is practically no difference in abrasion loss between the bottom and the side of specimen, whereas in the case of the air-entraining concrete, the abrasion loss on the side was practically twice that of the bottom.

Fig. 3 showing these same four concretes, presents an interesting study of the danger of not controlling air and cement content. It shows the top surfaces of the four concretes after the abrasion tests had been made. The differences are so great as to be easily noted merely by a visual inspection. The specimen on the left is a plain concrete having somewhat less than 1 per cent of air. The second specimen made with a straight

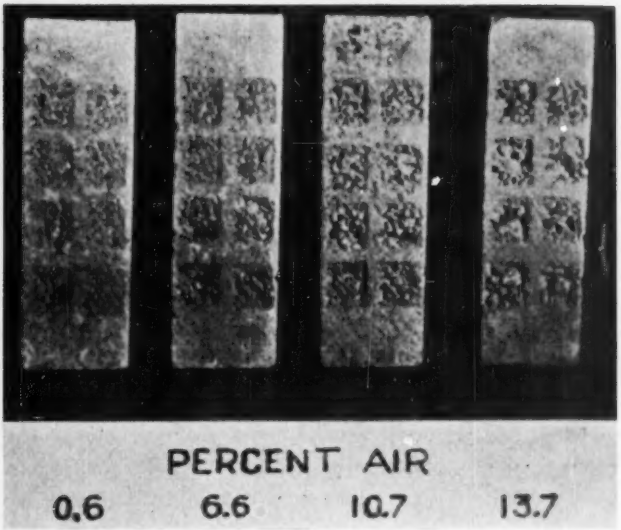

Fig. 3 — Abrasion tests of specimens with different percentages of air.

air-entraining agent contains 6 per cent, the third 10 per cent, and the last 13 per cent of air.

It will be noted that there is a progressive increase in the abrasion loss as the air content increases. It will also be noted that when the air content is kept below 6 per cent and cement content held constant, the abrasion loss is very little more than that obtained without the entrained air. It is suggested, however, that entrained air of over 10 per cent is actually dangerous from the standpoint of abrasion, even though the concrete would be quite remarkable from the standpoint of resistance to freezing and thawing.

Incidentally, this project involves approximately 1200 test specimens and one year's laboratory work. The specimens are being tested under various conditions of mix, cement content, air content and curing.

All of this work points the same way—do not be deceived by the very fatty (whip cream) mixes—be suspicious of these, because it has been demonstrated time and again that they can be an indication of excess air and may give concretes low in strength and abrasion resistance. Hold and control the air to a maximum of 6 per cent. Check and insist on maintenance of cement factor. The plasticity at this air content will be very satisfactory, as will be strength and other desirable properties.

To facilitate selective distribution, **separate prints** of this title (42-30) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946.

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Methods of Entraining Air in Concrete*

By E. W. SCRIPTURE, Jr.†
Member American Concrete Institute

SYNOPSIS

Discusses methods for and mechanisms of air entrainment in concrete mixes. Methods include use of aluminum and hydrogen peroxide for entrainment of hydrogen or oxygen, respectively; use of cement dispersing agents; perhaps protective colloids. Data are reported to record the relation of air content to durability as determined by freezing and thawing.

From a practical point of view it might not seem important to know the mechanism whereby air entrainment is effected, especially since it does not appear to make much difference, as far as durability is concerned, how the air is incorporated in the concrete. Hope for improvement of methods and materials for air entrainment and for control of this phenomenon, however, should lie in a knowledge of the mechanism or mechanisms. There appears to be a tendency to think only in terms of one means of air entrainment, that of employing surface active compounds which lower the surface tension of water, known variously as wetting or foaming agents, and including soaps. The mechanism whereby these compounds incorporate air in the concrete mix is well-known and has been described in several places in the literature. That this is the only air entrainment mechanism is manifestly incorrect and it is by no means certain that it is the most satisfactory.

Aluminum and hydrogen peroxide have been used to incorporate air, or rather hydrogen and oxygen, respectively, in concrete mixes. These function by generating the gases in situ by reaction with constituents of the cement and bear no relation to surface tension reducing compounds.

There is still a third known method of introducing air, above the normal amount, into concrete and that is by use of a cement dispersing

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Director of Research, The Master Builders Co., Cleveland, Ohio.

agent. These are surface active chemical compounds which are preferentially adsorbed by cement, endowing the cement particles with electrostatic charges which make them mutually repellent. These compounds do not lower the surface tension of water to a marked degree and do not form stable foams with water alone, although what they may do in a cement suspension is something else. They are definitely not wetting or foaming agents and would not be applicable to those uses of wetting or foaming agents which depend on surface tension reduction. The mechanism whereby they entrain air in concrete is evidently not the same as that of foaming agents although just what that mechanism is may be open to some question. It may be suggested that it is related to increased effective surface area of the cement and the finer effective size of the cement particles in the dispersed state.

While these three methods of entraining air are the best known, it by no means follows that they are the only ones. For example, the use of protective colloids may offer some possibilities. Solving the problem of air entrainment may lie in an understanding of how it works.

Undoubtedly cement dispersing agents normally entrain some additional air. A curious phenomenon in this connection is that, unlike the wetting agents, the amount of air entrained with a fixed ratio of dispersing agent to cement appears to be relatively constant regardless of the mix or the brand of cement used, within reasonable limits. Data on a series of 39 mixes using 13 different cements ranging from 4½ to 6 sacks of cement per cu. yd. and including slumps from 2 to 6 in. are shown in Table 1. It will be noted that for each type of mix the averages for the 13 cements range only from 3.1 per cent to 3.8 per cent of air. The extreme variation with the different cements and the different mixes is from 2.0 to 5.0 per cent but only 7 air contents are over 4 per cent and only 3 below 2.5 per cent, and only 7 others below 3.0 per cent. It may be doubted that the extreme values are significant. It is not suggested that

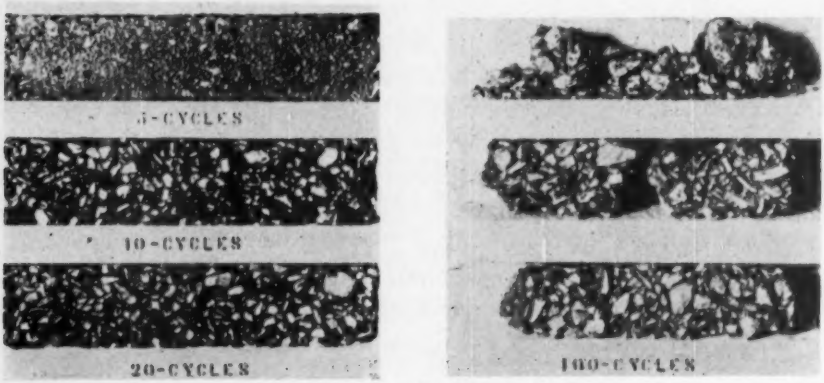

Fig. 1—Control Mix

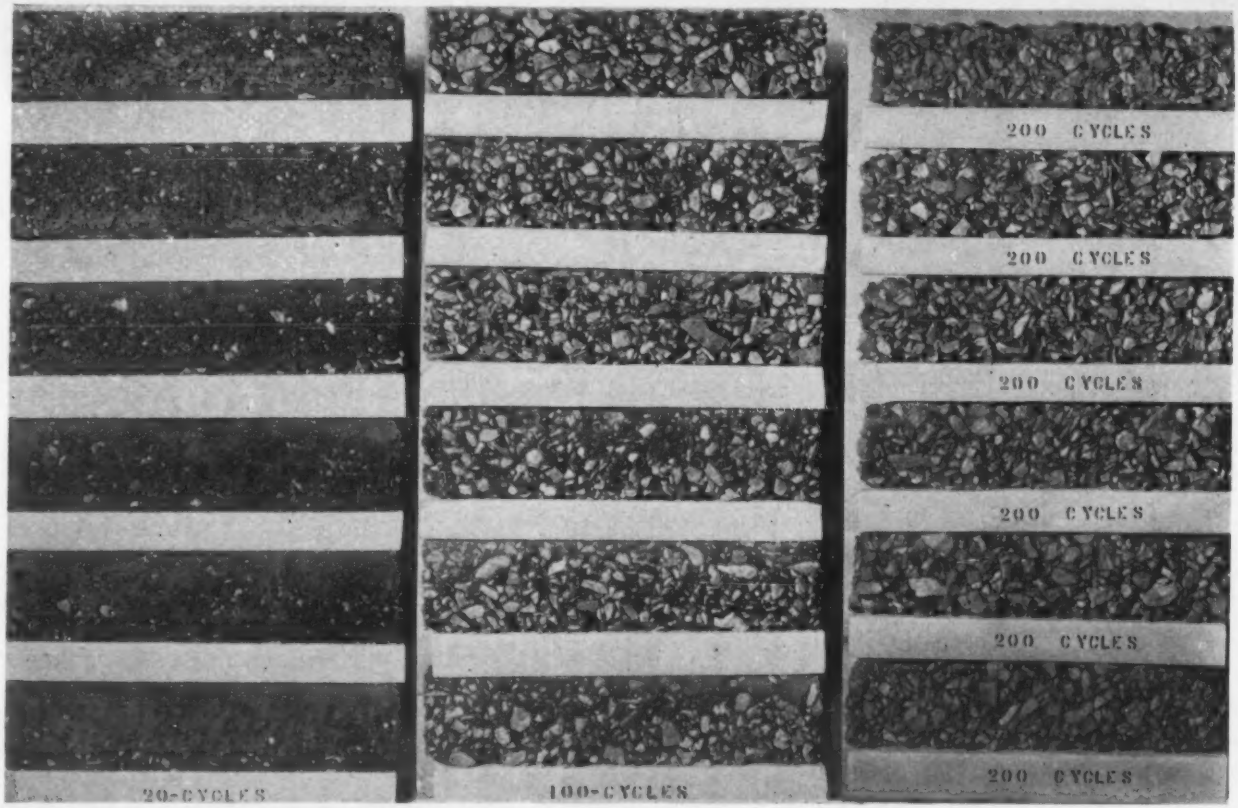

Fig. 2—3.5 per cent air—20 cycles. Fig. 3—3.5 per cent air—100 cycles Fig. 4—3.5 per cent air—200 cycles.

TABLE 1

Cement	Total Air Content—Per cent by Volume		
	6 Sack Nominal Mix		4½ Sack Nominal Mix
	Slump 3½ in.	Slump 6 in.	Slump 2 in.
A	3.7	4.0	3.5
B	3.5	3.6	2.9
C	3.8	3.7	3.8
D	3.7	4.6	3.4
E	2.8	3.1	2.0
F	2.7	3.2	2.9
G	3.3	4.4	4.3
H	2.4	2.8	3.0
I	3.3	3.7	2.9
J	3.6	4.4	4.5
K	2.7	3.0	2.4
L	3.9	3.8	3.5
M	3.8	5.0	4.1
Average	3.1	3.8	3.3

TABLE 2

Addition	S/A Ratio per cent	W/C gal. per sack	C.F. Sk. per cu. yd.	Total Air per cent	Comp. Str. 6 mo. Lb. per sq. in.
None	40	6.01	6.01	0.2	6870
Dispersing Agent + Foaming Agent	36.5	5.16	5.98	3.6	7265
Foaming Agent	36.5	5.61	5.95	3.4	6257
Dispersing Agent	36.5	5.20	6.00	3.5	7460
Dispersing Agent + Foaming Agent	40	5.36	5.97	3.5	7511
Foaming Agent	40	5.66	5.91	3.7	6270
Dispersing Agent	40	5.31	5.97	3.7	7612

it is not possible to vary the air content, especially by changing the sand-total aggregate ratio, when using a cement dispersing agent, but that with a fixed quantity of the cement dispersing agent per sack of cement a very constant air content, in the range generally considered most desirable, will be secured automatically in most normal mixes.

To determine the relation of durability to air content, regardless of the means used for air entrainment, a series of mixes of equal cement factors was made up to approximately constant air content, except for the control mix without addition, and subjected to freezing and thawing. The mix data, arranged in the order of illustrations, (Fig. 1 to 4) are given in Table 2. It may be of interest to note that the six-month strengths of the dispersing agent or dispersing plus foaming agent mixes average about 9 per cent higher than that of the control mix, although the air content has been increased from 0.2 to 3.5 per cent, whereas the foaming agent mixes exhibit about an equal loss in strength. The course of disintegration is shown in Fig. 1 for the control mix at 5, 10, 20, and 100 cycles and in Fig. 2, 3, and 4 for the mixes containing approximately 3.5 per cent air, however attained, at 20, 100, and 200 cycles.

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Effect of Air Entrainment on Stone Sand Concrete*

By A. T. GOLDBECK†
Member American Concrete Institute

SYNOPSIS

A limestone sand which had been used with indifferent success in pavements and other structures, prompted a series of tests by National Crushed Stone Association, to improve workability and durability of concrete in which this aggregate was used. Results with and without Vinsol resin as an air-entraining agent were favorable to the use of the admixture. Data reported include materials, mix design, freezing and thawing tests.

The following tests were made as a portion of a series of tests in an effort to improve the workability and durability of concrete made with a particular limestone sand which had been used in concrete structures and pavements with indifferent success. Previous tests included studies of the effect of changing the shape of particle, the effect of cement factor and investigations to determine the effect of the addition of different percentages of limestone dust.

The present report covers only what might be considered the successful portion of the investigation, namely, that involving the use of Vinsol resin as an air-entraining agent.

Six mixes were made; one with Potomac River sand with no Vinsol resin added, and five with limestone sand having Vinsol resin contents of 0.0, 0.009, 0.013, 0.017, and 0.025 per cent based on the weight of cement in the mixture. The Vinsol resin was dissolved in a 3 per cent solution of sodium hydroxide and this solution was added to the mixing water in the above amounts.

MATERIALS

Cement

One brand of type 2 cement was used, with a constant cement factor of 6 bags per cu. yd. in all of the mixes.

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Engineering Director, National Crushed Stone Association, Washington, D. C.

Sand

The natural sand used in one of the mixes for comparison purposes was from the Potomac River and its physical characteristics were as follows:

Bulk specific gravity 2.61
Weight per cu. ft. solid, lb. 162.9
Absorption, per cent 1.0
Gradation:

Sieve No.	Total per cent passing
4	100
8	85
16	75
30	50
50	15
100	4

Fineness Modulus = 2.76

The characteristics of the limestone sand were as follows:

Bulk specific gravity 2.64
Weight per cu. ft. solid, lb., ... 164.7
Absorption, per cent 1.1
Gradation:

Sieve No.	Total per cent passing
4	100
8	91
16	64
30	36
50	18
100	8

Fineness Modulus = 2.83

Shape of stone sand particle

A measure of the shape of stone sand particle was obtained in the manner described in N.C.S.A. "Useful Information" No. 2-28. It consists in determining the percentage of voids in a loose condition of the No. 8-16, No. 16-30, and No. 30-50 sieve fractions. The higher the percentage of voids, the more angular is the shape of particle. As determined by the above mentioned method, the percentage of voids equals 51.5. The indications are that this sand has a fairly cubical shape of particle.

When sand is prepared by special means, such as the Kent mill or the rod mill, it is possible to reduce the percentage of voids to about 50.0. On the other hand, stone screenings with no processing will ordinarily run about 53.0.

Coarse aggregate

A crushed limestone coarse aggregate with the following characteristics was used in these tests:

Source—West Virginia
Bulk specific gravity 2.72
Weight per cu. ft. solid, lb. 169.7
Weight per cu. ft. dry, rodded, lb. 103.7
Absorption, per cent 0.3

Gradation:

Sieves	Total per cent passing
2 in.	100
1½ in.	98
1 in.	67
¾ in.	52
½ in.	30
No. 4	2
No. 8	0

Design of concrete mixes

All mixes were designed in accordance with the method given in N.C.S.A. *Bulletin No. 11*, or *Stone Briefs No. 2*. The cement factor was held at 6 sacks per cu. yd. for all of the mixes and since the sand had a fineness modulus of about 2.80, the value for b/b_o, was taken at 0.77. An estimate was made of the volume of air entrained in the mixtures and this volume was considered as a component of the concrete. A slump of 2 to 3 in. was maintained.

Mixing, molding and testing

The concrete was mixed for 5 min. in a tilting drum, fixed blade type of mixer of approximately 3 cu. ft. capacity. One beam and two cylinders were cast from each mix. Twenty-four hours after molding, the specimens were removed from the molds and placed in the moist room where they were cured under standard conditions of moisture at 70 F temperatures until they were tested at the age of 28 days. Two compression and two flexural tests were made from each mix. Each beam was tested twice.

Freezing and thawing tests

Owing to the limited capacity of the freezing chamber, specimens were prepared from the broken ends of beams by sawing them to the dimensions of 3 x 3 x 12 inches. Gage plugs were set on 10-inch centers and a Whittemore strain gage was used to measure the permanent set after a number of alternations of freezing and thawing, so that curves could be drawn showing the expansion which took place as the result of freezing. Permanent expansion is a measure of the durability of concrete and it gives indications of durability parallel with those shown by dynamic modulus of elasticity. The specimens were kept completely immersed in water during both the freezing and the thawing periods.

Results of the tests

A summary showing the average of the three mixes is as shown in Table 1.

Fig. 1 shows the effect of different percentages of air entrainment on both the compressive strength and the modulus of rupture of the stone sand concrete. Fig. 2 shows the effect of Vinsol resin and air entrainment

TABLE I—AVERAGE VALUES OF THREE MIXES

Type of Fine Aggregate	Natural Potomac R. Sand	Stone Sand				
		A	B	C	D	E
Vinsol resin, percent	0.000	0.000	0.009	0.013	0.017	0.025
Parts per unit vol.						
Cement	.106	.106	.106	.107	.106	.107
Water	.156	.163	.158	.155	.151	.146
Air	.012	.011	.021	.026	.040	.065
Sand	.257	.250	.244	.239	.232	.210
Stone	.469	.470	.471	.473	.471	.472
Total	1.000	1.000	1.000	1.000	1.000	1.000
Slump, in.	2.6	2.5	2.4	2.4	2.5	2.8
Flow, per cent	71.	64.	61.	62.	69.	67.
Cement factor, sacks per cu. yd.	5.98	6.00	6.01	6.04	6.00	6.02
Entrained air, per cent	1.2	1.1	2.1	2.6	4.0	6.5
Sand, per cent	35.5	34.7	34.2	33.6	33.0	30.9
W/C by volume	.70	.73	.71	.69	.68	.66
Ratio (vol. water + vol. air to apparent vol. cement)	.76	.77	.81	.81	.86	.95
Compressive strength, psi	5950	5310	5250	5090	4890	4090
Modulus of rupture	860	870	830	810	805	685
Freezing and thawing tests (specimens from third set of mixes) Permanent expansion in. per in., 150 cycles	Failed	Failed	.00074	.00049	.00044	.00039

on the unit expansion which took place as the result of cycles of freezing and thawing.

The significant results of this series of tests can best be observed by studying the two charts. It will be noted that air entrainment produces a reduction in both the compressive strength and in the modulus of rupture. Assuming that the curves of strength shown in the chart fairly represent the average values, the effect of air entrainment on strength is shown in the following table.

Air	Compression		Modulus of Rupture	
0 per cent	5300	100 per cent	860	100 per cent
3 per cent	5000	94.5 per cent	840	98 per cent
5 per cent	4600	87 per cent	790	92 per cent

It will be noted that with 5 per cent of air entrainment which is frequently taken as a desired maximum, there is a reduction of 13 per cent in compressive strength and 8 per cent in modulus of rupture. The strength values in both compression and modulus of rupture are very high. Referring to the chart showing unit expansion with different cycles of freezing and thawing, it will be noted that all of the specimens containing air produced through the use of Vinsol resin, show very little expansion and, in fact, the expansion of specimen 3B with only 2.1 per cent of air in it, was no greater at 150 cycles than in the specimen containing no Vinsol resin, measured at less than 10 cycles.

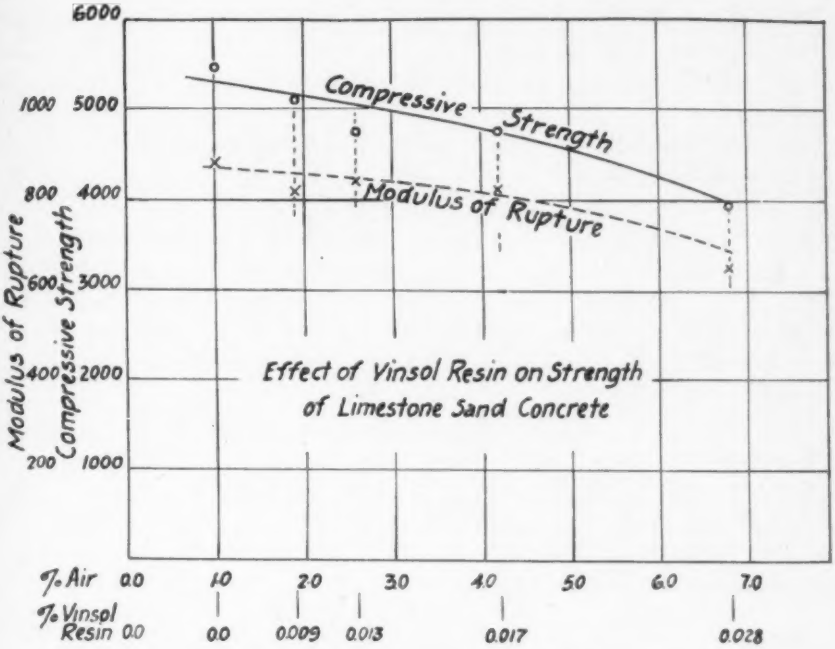

Fig. 1—Effect of Vinsol resin on strength of limestone sand concrete

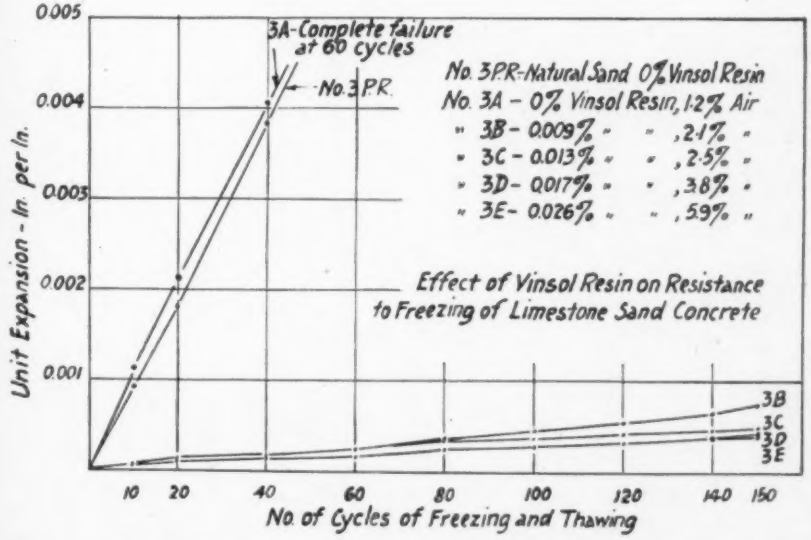
3A-Complete failure at 60 cycles
No. 3P.R.
No. 3P.R-Natural Sand 0% Vinsol Resin
No. 3A – 0% Vinsol Resin, 1.2% Air
" 3B – 0.009% " " , 2.1% "
" 3C – 0.013% " " , 2.5% "
" 3D – 0.017% " " , 3.8% "
" 3E – 0.026% " " , 5.9% "
Effect of Vinsol Resin on Resistance to Freezing of Limestone Sand Concrete
Unit Expansion - In. per In.
0.005
0.004
0.003
0.002
0.001
3B
3C
3D
3E
10 20 40 60 80 100 120 140 150
No. of Cycles of Freezing and Thawing

Fig. 2—Effect of Vinsol resin on resistance to freezing of limestone sand concrete

The effect of the Vinsol resin inclusion on the workability of the concrete was extremely marked. Even specimen 3B, with only 2.1 per cent of air and the minimum amount of Vinsol resin, was very plastic and, furthermore, this mix showed practically no water-gain. This was in marked contrast with specimen 3A which contained no Vinsol resin. All of the specimens showed surface scaling at the end of 150 cycles, but the specimens containing no Vinsol resin were unsound as indicated by their partial breaking apart and by the looseness of the coarse aggregate in the matrix. Furthermore, they had a very dead sound when struck with a hammer. On the other hand, all of the specimens containing Vinsol resin were in an entirely sound condition. The coarse aggregate was tightly bound and when struck with a hammer the specimens had a metallic ring.

Air entrainment evidently will not entirely prevent surface scaling of concrete upon severe exposure, but it does effect a very material improvement in the durability of concrete and in its resistance to scaling. It is believed that these tests indicate very strongly that the limestone sand used will make very durable concrete, provided from 3 to 5 per cent of air is entrained by the use of an air-entraining agent.

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A Method for Direct Measurement of Entrained Air in Concrete*

By W. H. KLEIN† and STANTON WALKER‡
Members American Concrete Institute

SYNOPSIS

Since the amount of air entrained in concrete is of major importance and the methods now in use to determine that amount have inherent objections, the pressure method, by application of Boyle's law, has advantages. The Klein air meter is described, following tests by Pennsylvania-Dixie Cement Corp. and National Ready Mixed Concrete Association. The paper presents a description, with illustrations, of the Klein air meter, the test procedure, calculation of air content and calibration of meter and presentation of data on the use of the method.

The amount of air in concrete is of major importance when air-entraining agents are used. If too little is entrained the potential benefits are not realized; if too much, serious reductions in strength and other disadvantages result. Methods in general use for the determination of the quantity of entrained air in concrete have inherent in them certain objections—either from the points of view of accuracy or of convenience. This paper describes a procedure for the direct measurement of air which offers promise of overcoming some of these objections.

Two general methods for measuring air in concrete have been used. The most convenient procedure, and the one best adapted to field use, is patterned after that described in A.S.T.M. Method C 138—*Standard Method of Test for Weight per Cubic Foot, Yield, and Air Content (Gravimetric) of Concrete.* By it, the air-content is calculated from knowledge of the weight per unit of volume of the concrete and the quantities and specific gravities of the ingredients. The second method depends on the determination of the weight of concrete per unit of volume and on the displacement in water of a weighed sample of the concrete, after elimi-

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Vice-President, Pennsylvania-Dixie Cement Corp.
‡Director of Engineering, National Ready Mixed Concrete Association.

nation of the entrained air from the sample while immersed in water. A specific procedure applying that principle is described in A.S.T.M. Method C 173, *Tentative Method of Test for Air Content (Volumetric) of Freshly Mixed Concrete.* The method described herein permits direct determination of air content without the necessity of knowledge of any characteristics of the batch.

The gravimetric method depends upon the accuracy with which the required data are known. If the proportions of ingredients differ from those assumed, an error is introduced; a principal source of error is lack of precise knowledge of the specific gravities and moisture content of the ingredients of the batch. Small errors in these factors lead to relatively large errors in the results.

The volumetric method is tedious of execution and, in addition to common errors of measurement, is subject to such errors as may result from failure to remove all of the entrained air. The authors believe it, however, to be basically sound.

PRESSURE METHOD FOR DETERMINING AIR

The method described in this paper depends on a simple application of Boyle's Law. Neither the weight per unit of volume nor the proportions or specific gravities of the ingredients are required. Pressure is applied to a known volume of concrete and the reduction in volume measured. Since the air entrained in the concrete is the only significantly compressible ingredient, whatever reduction in volume results is due to its compression. From a knowledge of the pressure applied and the reduction in volume, the amount of air can be calculated readily.

The idea for the procedure originated with the senior author. On being discussed with the junior author it aroused his interest and the desire to investigate its possibilities. Exploratory work was done in both the Central Laboratory of the Pennsylvania-Dixie Cement Corporation, Bath, Pa., and the Research Laboratory of the National Ready Mixed Concrete Association at the University of Maryland, College Park, Md. At Bath, the work was done by L. C. Hawks, Chief Chemist; at Maryland by D. L. Bloem, Associate Research Engineer.

At the Central Laboratory of the Pennsylvania-Dixie Cement Corporation an apparatus of one liter capacity was constructed. Pressure-method tests were made on mortar and, also, on concrete with aggregate up to ¾ in. maximum size. For mortar, results were compared with determinations made in accordance with Tentative Method of Test for Air Content of Portland Cement Mortar (A.S.T.M. Designation, C 185-44T); for concrete the comparisons were with Tentative Method of Test for Air Content (Volumetric) of Freshly Mixed Concrete (A.S.T.M. Designation, C 173-42T). Satisfactory agreement was found in both

cases. A large number of routine tests have been made comparing the pressure method with the two standard methods, also with satisfactory agreement.

At the University of Maryland tests were conducted with improvised equipment and with especially constructed apparatus of one-half cu. ft. capacity, suitable for concrete, and described later. Comparisons were made with the gravimetric method with satisfactory results. The apparatus and tests described in this paper are based on the work done at the University of Maryland.

The tests conducted were not extensive. However, they were adequate to convince the authors that the method offers definite promise and deserves of description to permit others to compare it with currently used procedures. It is hoped that such work may lead to a standardized procedure which will serve, at the least, as a basis for "calibrating" the convenient gravimetric field method.

THE KLEIN AIR METER

Fig. 1 is a schematic layout of the appratus used in the investigation. It consists essentially of a source of air pressure (the Pressovac pump), a pressure gauge (the mercury manometer) and a container identified as the Klein Air Meter. The "surge tank," consisting of a standard glass carboy, is incidental, being required because of the over-capacity pump used. In the field, pressure would be applied by some such simple device as an ordinary tire pump, which has been used with complete satisfaction. For the pressure gauge, there would be substituted for the somewhat clumsy manometer one of several standard dial gauges of suitable sensitivity, which are readily available on the market.

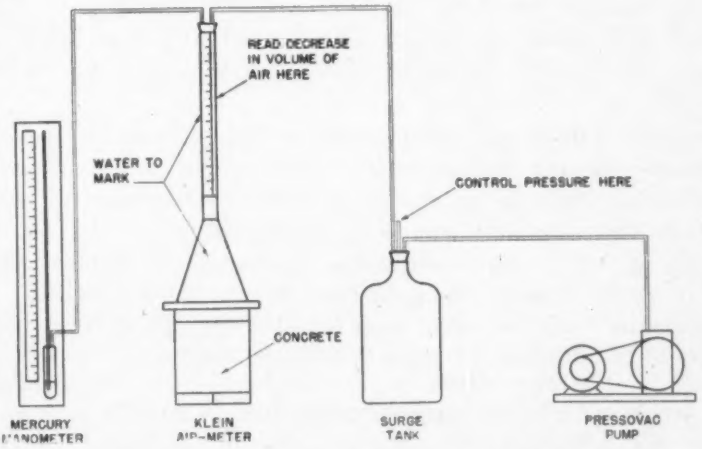

Fig. 1.—Schematic layout of Klein Air-Meter in operation in laboratory.

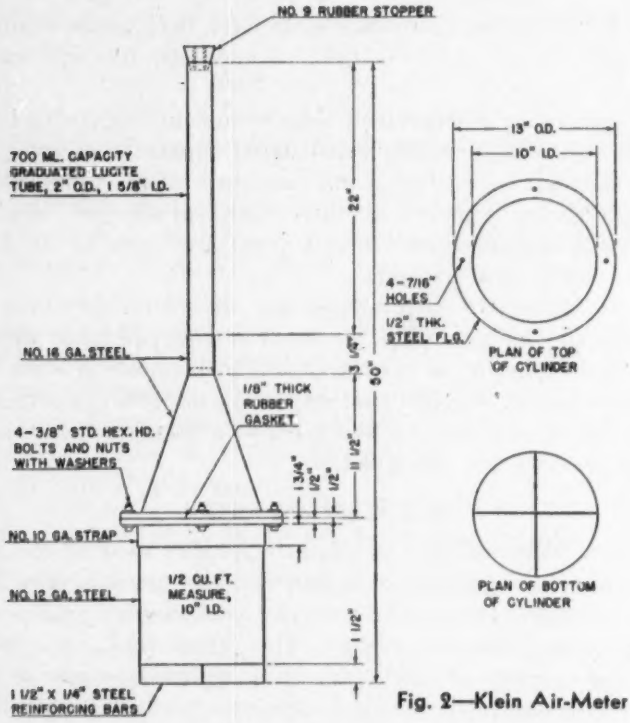

Fig. 2—Klein Air-Meter

Fig. 2 shows the essential details of the air meter. The one illustrated is a purely experimental model manufactured under the authors' direction by C. A. Hogentogler, Jr. It lacks a number of refinements which would be built into any subsequent model. It consists of a standard ½ cu. ft. measure (see A.S.T.M. Method C 29) on which an assembly consisting of a metal truncated cone equipped with a graduated lucite tube is mounted, with a suitable rubber gasket to make the connection water-tight.

Among the refinements which would be incorporated in later models are a drain-cock near the base of the truncated cone and a threaded top, with suitable outlets, to replace the indicated rubber stopper. A source of pressure and a pressure gauge would also be furnished. Further, it seems likely that a more convenient method for attaching the cone assembly to the measure, than the bolts shown, can be devised. While this apparatus would permit of more detailed description, what has been said should be sufficient to make clear its fundamental features.

TESTING PROCEDURE

The procedure for making the test may be described briefly as follows:

(1) Fill the ½ cu. ft. measure with concrete in a standard fashion (see A.S.T.M. Method C 138).

(2) To avoid turbulence (the source of which will become clear later) place on top of the concrete a glass plate or a thin rubber sheet, or some similar device, which is slightly less in diameter than the measure.

(3) Attach the cone and graduated tube assembly in such a manner as to be assured that the connection is water-tight.

(4) Introduce water into the cone assembly to the zero mark on the graduated tube. To avoid turbulence and the inclusion of casual air, the water should be introduced slowly through a hose, the end of which is near the covered surface of the concrete in the container. (In a later model, the water would probably be introduced through the drain cock).

(5) Cap the tube with a stopper having two outlets, one to a source of pressure and the other to a pressure gauge.

(6) Apply pressure and read the amount of pressure applied and the reduction of volume of water in the graduated tube. (In our practice, pressures have been restricted to various values up to about 40 centimeters of mercury, or a little more than ½ atmosphere).

That is all there is to the testing procedure, and experience with it demonstrates that it works for a wide range of mortar and concrete mixtures. Typical data will be discussed later.

CALCULATION OF AIR CONTENT

As stated previously, from a knowledge of the reduction in volume, as determined by reading the graduated tube, the volume of air can be calculated by the straight-forward application of Boyle's Law. With certain minor reservations of no interest here, that law is that the volume occupied by a given mass of any gas at constant temperature varies inversely as the absolute pressure to which it is subjected. Accordingly we can write,

$$\frac{V}{V-r} = \frac{P_2}{P_1} \qquad (1)$$

where,

V = volume of air in the concrete
r = reduction in volume due to pressure
P_1 = initial absolute pressure
P_2 = final absolute pressure

The initial pressure consists of the barometric pressure plus pressure incidental to the procedure and the final pressure consists of the gauge pressure plus the initial pressure and other pressure incidental to the procedure. It seems worth while to derive the applicable formulae; for that purpose assume the following nomenclature:

V_1 = Volume of air in fresh concrete in container.
V_2 = Volume of air in fresh concrete in container after water has been brought to mark in cone assembly.
V_3 = Volume of air in fresh concrete in container after application of pressure.
V_4 = Volume of concrete in container.
h_1 = Average pressure exerted by concrete (taken as head of concrete to mid-height of measure).
h_2 = Pressure exerted by column of water brought to mark in cone assembly.
h_3 = Pressure exerted by column of water in cone assembly after application of pressure (h_3 is less than h_2, closely enough, by the amount which the height of the column of water is reduced in the calibrated tube when pressure is applied.)
B = Barometric pressure.
P = Gauge reading on application of pressure.
R = Reduction in volume of water in calibrated tube on application of pressure P.

Then,

$$\frac{V_1}{V_2} = \frac{B + h_1 + h_2}{B + h_1} \quad \ldots\ldots (2)$$

$$\frac{V_2}{V_3} = \frac{P + B + h_1 + h_3}{B + h_1 + h_2} \quad \ldots\ldots (3)$$

$$V_3 = V_2 - R \quad \ldots\ldots (4)$$

Combining equations 2, 3, and 4, we have,

$$V_1 = \frac{R(B + h_1 + h_2)\,(P + B + h_1 + h_3)}{(B + h_1)\,(P - h_2 + h_3)} \quad \ldots\ldots (5)$$

The percentage of air is, of course,

$$A = 100\,\frac{V_1}{V_4} \quad \ldots\ldots (6)$$

CALIBRATION OF AIR METER

While barometric pressures vary sufficiently from locality to locality (for example, New York to Denver) so that they should be taken into account, the variation in any one general locality is not likely to be sufficient to cause appreciable error. Consequently, if a mean barometric pressure is assumed and an applied gauge pressure is selected, the tube may be calibrated to read percentages of air directly. Further, this same calibration can be made to apply for other barometric pressures without appreciable error if appropriate applied pressures are selected.

Equations 5 and 6 can be combined and written,

$$R = \frac{A\,V_4\,(B + h_1)\,(P - h_2 + h_3)}{100\,(B + h_1 + h_3)\,(P + B + h_1 + h_3)} \quad \ldots\ldots (7)$$

$$A = \frac{100R}{V_4}\,\frac{(B + h_1 + h_2)}{(B + h_1)}\,\frac{(P + B + h_1 + h_3)}{(P - h_2 + h_3)} \quad \ldots\ldots (8)$$

Equation 8 has been written to indicate the most convenient subdivision of steps in making calculations. Knowing the cross sectional area of the tube (C), R can be expressed in terms of h_2 and h_3. For illustration, expressing all dimensions in centimeters and all pressures in terms of mercury, which has a specific gravity of 13.6, we have,

$$R = 13.6\,C\,(h_2 - h_3) \quad \ldots\ldots (9)$$

From a knowledge of the dimensions of the air meter, R can be calculated for selected percentages of air A for fixed values of P and B, and

thus the graduations on the tube can be made to show percentages of air directly. For any other barometric pressure B than that assumed in the preceding, calculations based on these equations will show that the same calibration can be maintained, without significant error, by using another value for gauge pressure P.

At the risk of belaboring this point, let us, for illustration, calculate R for this apparatus for 5 and 10 per cent of air, when the applied gauge pressure is 40 cm. and the barometric pressure is, closely enough, 76 cm. of mercury. The volume of the measure is ½ cu. ft., or 14160 cu. cm. The weight per cu. ft. of the concrete will not vary enough to have a significant effect and, assuming it at 145 lb., h_1, for the dimensions shown, becomes 2.5 cm. of mercury. Pressure h_2 depends on the position of the "zero mark" (not shown on the drawning) and, for the apparatus used, is 6.6 cm. of mercury.

The application of equations 7 or 8 and 9 shows R to be 204 cu. cm. for 5 per cent of air and 402 cu. cm. for 10 per cent, when the barometric pressure is 76 cm. and the applied pressure is 40 cm. of mercury. From these same equations it can be found that a reduction in volume of 204 c.c. can also be made to indicate 5 per cent of air for other barometric pressures if the correct applied pressure is used. For a barometer of 60 cm., the application of 33.8 cm. of pressure will accomplish that result. Calculation will show that the application of this same pressure to concrete containing 10 per cent of air will give the correct result within less than 0.1 per cent.

Such a refinement in calibration as described above is intriguing but somewhat academic. It probably is not worth the trouble, and, moreover, it detracts from flexibility in use of the meter; it ties the user to one applied pressure. The authors believe the advantages and disadvantages balance in favor of calibrating the tube in terms of volume—most conveniently, cubic centimeters.

The computations need not be laborious. Equation 8, combined with equation 9, is so nearly a straight line, when plotted, that a simple table can be constructed which gives factors which may be used to calculate the percentage of air by the simple division of the reduction in volume R by the factor K. Table 1 shows factors for this apparatus, assuming a barometer of 76 cm. and an applied pressure of 40 cm. of mercury. Similar factors can be calculated readily for other barometric and applied pressures.

To illustrate the use of the table, consider a reduction in volume R of 225 c.c. The factor for 200 will represent adequate accuracy. The per cent of air would be $\frac{225}{40.9} = 5.5$ per cent. The precise application of

TABLE 1—FACTORS FOR COMPUTING AIR CONTENT

Per cent air $= \frac{R}{K}$, where R = reduction in volume of water column in cu. cm. and K = factor for barometric pressure of 76 cm. and applied pressure of 40 cm. of mercury.

Using the value of "K" nearest to reading "R" will yield percentages accurate to within 0.02. If greater accuracy is desired, other values for "K" may be obtained by interpolation.

Reduction in Volume (R, cu. cm.)	Factor K
100	41.5
200	40.9
300	40.4
400	40.3
500	39.9
600	39.6
700	39.1

the basic equation would permit of no different reading on a 20 in. slide rule.

TEST DATA

The operation of the air meter is relatively simple. Some troubles are encountered with casual air introduced with the water, but they can be minimized by using water that is relatively air free and by exercising care in filling the cone assembly. When air is included with the water it should be removed by gentle tapping of the container. If vigorous methods are used, air in the concrete also will be displaced.

Relatively few tests suitable for reporting have been conducted although many preliminary trials were made in developing technic. A typical group of data, for concrete having a wide range in air content, are shown in Table 2. The concrete contained 5.5 sacks of cement per cu. yd. and had a slump of 3 to 4 in.; well-graded sand and gravel with a maximum size of 1 in. was the aggregate. The specific gravities of the ingredients were known with accuracy and the results of the air meter are compared with those by the gravimetric method (i.e., calculated from absolute volumes).

With the air meter, it will be observed, 10 to 15 readings were taken for each determination. It should not be assumed that such detail is suggested for standard use. The procedure was to take readings at applied pressures of 20, 30, and 40 cm. of mercury both as the pressure was applied and released, and to repeat this operation one or two times. In all cases the water in the column did not return to the zero mark and, before starting the next run, water was added to mark.

The results for the air meter are, with one exception, higher than those arrived at by computations based on the gravimetric method, the differences averaging about 0.6 per cent of air. These differences are

TABLE 2—DETERMINATIONS OF ENTRAINED AIR IN CONCRETE BY AIR METER AND GRAVIMETRIC METHODS

Klein Air Meter illustrated in Fig. 2 used; weight per cu. ft. determined in the measure forming part of the meter.

Concrete contained 5.5 sacks of cement per cu. yd.; slump of 3 to 4 in.; aggregate, well graded sand and gravel with maximum size of 1 in.

Concrete mixed six minutes in tilting mixer of 1 cu. ft. capacity.

Applied pressure cm. of *Hg.*	Air Contents for Different Batches						
	1 (0.000)*	2 (0.010)	3 (0.010)	4 (0.015)	5 (0.015)	6 (0.020)	7 0.020)
				1st Run			
20	2.0	7.3	6.8	9.8	8.9	11.2	12.1
30	2.1	7.2	6.8	9.9	8.8	10.8	12.2
40	2.1	7.1	6.8	9.7	8.6	10.8	12.6
30	2.1	7.3	6.8	9.9	9.1	11.6	12.4
20	2.1	7.3	6.8	9.9	9.1	11.5	12.1
Average	2.1	7.2	6.8	9.8	8.9	11.2	12.3
				2nd Run			
20	2.1	7.2	6.8	9.8	9.2	11.4	12.0
30	2.1	7.2	6.8	9.9	9.2	11.4	12.0
40	2.1	7.1	6.8	9.5	8.8	11.0	12.1
30	2.0	7.2	...	9.9	9.2	11.4	12.9
20	2.1	7.3	...	9.9	9.0	11.4	12.2
Average	2.1	7.2	6.8	9.8	9.1	11.3	12.2
				3rd Run			
20	2.1	7.1	6.9	9.8	9.1	11.5	...
30	2.1	7.2	6.7	9.9	9.1	11.4	...
40	2.1	7.2	6.8	9.4	8.8	11.1	...
30	2.1	7.3	6.8	9.9	9.1	11.4	...
20	2.1	7.3	6.8	9.9	9.1	11.4	...
Average	2.1	7.2	6.8	9.8	9.0	11.4	...
Grand Average	2.1	7.2	6.8	9.8	9.0	11.3	12.2
Gravimetric Method	1.3	6.5	6.1	8.6	9.1	11.2	11.8

*Vinsol resin per cent by weight of cement.

consistent with those found by others between the gravimetric and volumetric methods; J. C. Pearson, in his paper in the 1944 *Proceedings* of the American Society for Testing Materials, reported an average difference between the two methods of 0.8 per cent.

A study of the range in readings for each determination will give an idea of the reproducibility of the results. For reference numbers 1, 2 and 3, with air contents of less than 7 per cent, the variations are negligible, generally being of the order of 0.1 per cent. For reference numbers 4, 5, 6 and 7, with air contents of 9 to 12 per cent, somewhat greater variations were experienced, but none excessive. Experiences suggest that, with a later model of the air meter, even these small variations would be reduced. It is noteworthy that determinations at applied pressures of 20 and 30 cm. of mercury appear to be at least as reliable as those at 40 cm. There are indications that the use of the lower pressures is preferable.

Prior to the construction of the air meter, tests of the air content of mortar were made by adaptation of a flask (the Montreal Moisture Meter), designed for determinations of specific gravity and surface moisture of aggregates. The results of these preliminary tests are shown in Table 3. Here again it will be observed that the air contents shown by the pressure method differ from those by the gravimetric method in about the same degree as has been found for the volumetric procedure.

The question has been raised as to whether or not this procedure is applicable in the case of extremely harsh mixtures. In an endeavor to answer it, some tests have been made, only recently and incompletely, with under-sanded concrete using blast-furnace slag as a coarse aggregate. Experiences revealed no particular difficulties due to the harshness of the mix. The results with the air meter were in all cases higher than those arrived at by computation.

TABLE 3—DETERMINATIONS OF ENTRAINED AIR IN MORTAR BY PRESSURE AND GRAVIMETRIC METHODS

Weights per unit of volume measured in a 520 cc. container.

Determinations by pressure method made in Montreal Moisture Flask containing 1475 cc. of mortar.

Mortar 1 part cement to 2 parts sand by weight using Standard No. 20 to 30 Ottawa sand. W/C = 0.6 by weight. Vinsol resin in NaOH solution used as air-entraining agent.

Test No.	Percent Air	
	By Unit Weight Method	By Pressure Method
1	12.0	13.0
2	10.2	11.5
3	11.6	12.0
4	8.7	9.1
5	11.5	12.1
6	10.0	10.3
7 (Sand only)	24.7	25.7

However, a complication was encountered, probably because of the porosity of the slag. The data are shown in Table 4. The spreads between determinations by the air meter and by the gravimetric method were considerably greater than for the sand and gravel concrete. Comparing the "grand average" air meter results with the gravimetric

TABLE 4—DETERMINATIONS OF ENTRAINED AIR IN SLAG CONCRETE BY PRESSURE AND GRAVIMETRIC METHODS

Klein Air Meter illustrated in Fig. 2 used; weight per cu. ft. determined in the measure forming part of the meter.

Concrete contained 4.4 sacks of cement per cu. yd.; slump about 1 in.; aggregate, sand and blast furnace slag with maximum size of 1 in.

Concrete mixed six minutes in tilting mixer of 1 cu. ft. capacity.

Applied Pressure cm. of *Hg.*	Air Contents for Different Batches			
	1 (0.010)* (52)†	2 (0.020) (51)	3 (0.000) (40)	4 (0.015) (36)
1st Run				
20	9.9	13.7	3.0	10.1
30	10.2	14.0	3.7	11.8
40	9.9	...	5.2	11.9
30	11.4	...	4.0	12.9
20	12.0	15.9	4.5	13.6
Average	10.7	14.5	4.1	12.1
2nd Run				
20	11.2	15.1	4.5	12.6
30	11.0	14.5	4.6	12.5
40	10.6	...	...	12.5
30	11.8	...	...	12.9
20	12.0	16.1	...	13.1
Average	11.3	15.2	4.6	12.7
3rd Run				
20	11.2	15.1	4.4	12.5
30	11.1	14.7	4.6	12.7
40	10.6	...	4.5	12.6
30	11.6	...	4.6	13.1
20	11.4	16.0	4.9	13.4
Average	11.2	15.3	4.6	12.9
Grand Average	11.1	15.0	4.4	12.6
Gravimetric Method	8.1	12.9	2.5	9.0

*Vinsol resin per cent by weight of cement.
†Sand, per cent of total aggregate.

results, the average spread was 2.8 per cent as compared with the 0.6 per cent stated earlier. A part of this discrepancy may be accounted for (not surely) by inadequate knowledge of the specific gravity of the slag—a determination difficult to make with assurance. The complication referred to is represented by the progressively larger amounts of air found in the same sample by successive determinations, and the lower amount generally revealed by the highest pressure used.

The data are presented as a matter of interest and for whatever they may be worth. The need for additional work is recognized, particularly with lower air contents. However, explanations, which appear logical, present themselves for the apparent inconsistencies. The slag, in a dry condition, was soaked in water for 24 hours before being incorporated in the concrete. Obviously, that was not adequate time for complete saturation. It is reasoned, therefore, that as pressure was applied water was forced into the slag with the release of a corresponding amount of air to the concrete, revealed by subsequent determinations. That some such phenomena occurred is confirmed by tests made on slag alone, with the material dry, soaked for 24 hours, and saturated under a vacuum—with the first two indicating the presence of air and the latter showing no air on any run.

The lower value of air (sometimes) for the highest pressure might be reasoned as resulting from particle interference of the aggregates.

Both of these "complications" suggest the desirability of using a lower pressure, particularly with non-saturated porous aggregates, to minimize the additional absorption and, also perhaps, to minimize the effects of particle interference.

CLOSING REMARKS

The method for the direct measurement of the amount of entrained air in concrete described in this paper is recommended as being worthy of favorable consideration. The basic theory is sound. Experiences with the method convince the authors that the application of the theory is workable. The relatively few data cited are convincing; even more convincing are the preliminary tests made by both authors in developing the technic. The authors' excuse for reporting the procedure without complete investigation is their desire to make it available to other investigators at an earlier date than would otherwise be possible.

No rights are reserved by the authors with respect to the use of the procedure or the device described. However, with respect to the device, they wish to acknowledge assistance received from C. A. Hogentogler, Jr., who constructed the one used in most of the tests and who has since built a more refined model.

To facilitate selective distribution, **separate prints** of this title (42-33) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946.

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Automatic Dispensing Equipment for Air-Entraining Agents*

By R. R. KAUFMAN†
Member American Concrete Institute

SYNOPSIS

Automatic dispensing equipment is described as a means of getting an admixture into the concrete mix, at the mixer, with all the accuracy desirable.

Others have indicated the great improvements in concrete structures resulting from the use of certain admixtures and the need to introduce such admixtures uniformly at the mixer if desired results are to be achieved. The desirability of utilizing automatic dispensing equipment for this purpose has long been recognized by our company and we devised and used such equipment over five years ago.

The control of air entrainment and the other properties of the concrete is extremely difficult unless the addition is made at the mixer and in variable quantities. The addition should be at least as automatic and accurate as the measurement of any other constituent of the mix. To meet this need we have perfected a dispensing device which is accurate, rugged, easily regulable, and automatic.

Our problem in designing dispensing equipment was to provide an apparatus which would be rugged and easily regulable for different batches with varying cement contents over a wide range—from ½ yd. to 6 yds.—one which was at least as accurate as the other batching devices used in batching or ready-mix plant operation for sand, aggregate and cement.

In our experimental installations we found meters or other apparatus now available for this purpose wholly inadequate and unsatisfactory. Frequent plugging of the equipment by foreign materials such as cement and sand occurs. Equipment which depends upon close clearances not

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Chief Engineer, The Master Builders Co., Cleveland, Ohio.

only becomes easily plugged but soon shows excessive wear, rendering the equipment inaccurate after a period of operation under conditions encountered in batching and ready-mix plants.

Fig. 1 shows the general arrangement of the installation of the automatic dispensing device at the batching plant. It consists of storage tanks, pump and automatic dispenser. The storage tanks for the cement dispersing agent or other type of air-entraining agent in liquid form are usually located at ground level. Two tanks are generally provided so that one can be filled while the other is being used, resulting in continuous operation of the batching plant. The cement dispersing agent solution is of such concentration that 2 quarts are required per bag of cement. Solutions of foaming type air-entraining agents likewise are used in similar concentration because these relatively dilute solutions lend accuracy to the operation in contrast to the measurement of the amount in cubic centimeters. A pump connected to the storage tanks elevates the solution to the automatic dispensing unit. This is located at any place convenient for the batching plant operator.

The automatic dispensing unit is a rugged instrument built for heavy duty service. It has a large, easily readable dial calibrated in pounds as well as in bags of cement. It is only necessary to set the dial to correspond to the amount of cement used in the mix; then when the start button is pressed the dispenser will deliver the required amount of the reagent solution for the batch.

A 2-in. pipe connected to the outlet of the dispensing unit conducts the solution to desired delivery point. In central mix plants delivery of the solution is generally direct to the concrete mixer, while in transit mix operations it is usually into the mixer trucks. When mixing starts, practically immediately after charging either the mixer or the truck equipment, there is no measurable variation in results regardless of the sequence in which the reagent solution and other constituents of the mix are discharged. If the operation is so conducted that mixing is deferred until arrival at the job site, then the reagent solution is discharged with the sand and aggregate, and the cement is discharged last so that it is on the top of the load. When dry batching into compartment trucks from a central batching plant, the reagent solution is discharged simultaneously with the sand and aggregate and the cement is batched on top of the aggregate.

Fig. 2 is a view looking into the top of the dispensing unit. The construction is extremely simple and very accessible. It consists of a calibrated tank with a constant level overflow and an inverted weir which is regulable by the knob in front of the dial.

The automatic operating mechanism of the dispensing unit is shown in Fig. 3. When the start button is pushed it causes the discharge solenoid

valve to open and simultaneously starts the timer. This allows the reagent solution to flow out of the dispenser until it reaches the level

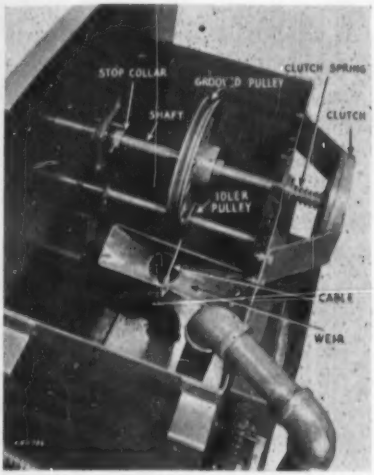

Fig. 1 (right)—Diagramatic sketch of Model A Pozzolith dispensing device installation.

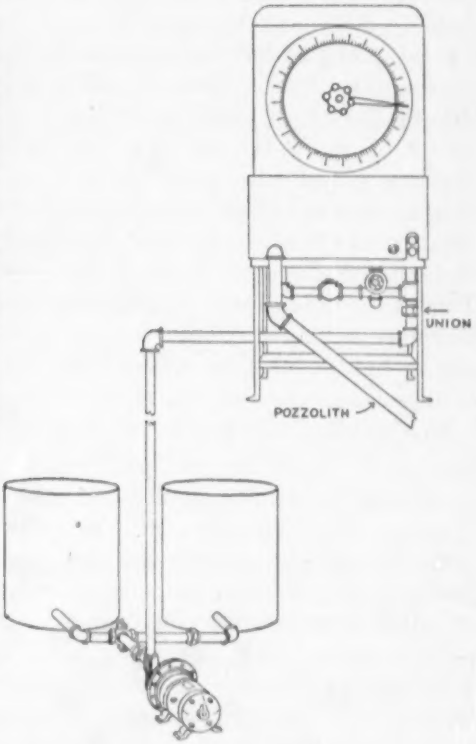

Fig. 2 (above)—Interior of the dispensing unit.

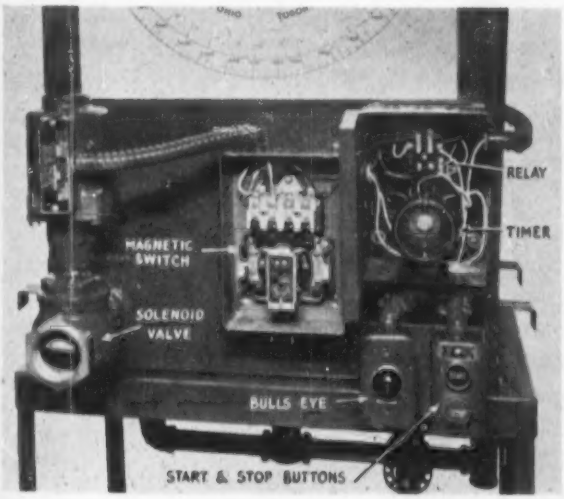

Fig. 3—Dispenser operating equipment.

of the top of the weir shown in Fig. 2. The time that the valve is open is determined by the setting of the timer. Normally 30 seconds is sufficient to empty the entire capacity of the dispenser. If the dispenser is to be used to dispense the reagent into dry batch trucks having charges of 1 cu. yd. and carrying two or more charges per truck, the timer may be set for 10 to 12 seconds which will provide ample time for the dispenser to deliver the small quantity of liquid required. After the time cycle, corresponding to the setting on the timer, has elapsed the solenoid valve closes automatically, starting the motor and pump connected to the storage tanks. The pump rapidly fills the dispenser with the reagent solution to the constant level overflow located at the back. Overflow of the solution opens the circuit on the overflow switch and stops the pump. During the entire cycle of operation the red bulls eye on the front of the dispenser is lighted. This indicates when the equipment is in operation and, if lighted after the normal time cycle, it usually indicates that the storage tank is nearly empty.

The dispenser has a dial calibrated both in pounds and bags of cement to make it easy for the operator to set the instrument without calculations to deliver the proper quantity of reagent solution. This arrangement is satisfactory for cement dispersion where the same amount is required per sack of cement regardless of the amount of cement per cu. yd., slump, design of mix, or other variable properties. In the case of foaming type air-entraining reagents which require variable quantities to produce the desired results, the dispensing unit has a variquantity attachment. When the variquantity attachment is used, the quantity of the foaming type air-entraining reagent may be varied from 66⅔ per cent to 133 per cent of the standard amount. The solution in the storage tanks is made of such concentration that 1½ quarts of the solution contain the predetermined amount of the reagent to produce correct results under controlled standard conditions. The variquantity attachment is then reset by the concrete technician from time to time as materials or conditions vary on the job. After the concrete technician re-sets the variquantity attachment then it is necessary only for the batch plant operator to set the dial of the dispenser for the pounds of cement used in the mix and push the start button to discharge the proper amount of reagent solution into the concrete mix. The dispenser may be equipped with an autographic recording device.

In some more modern batching and ready-mix plants which use automatic dial scale equipment for proportioning their mixes, it is possible to connect and synchronize the automatic dispensing unit with the cement scale.

Automatic dispensing units are now installed in more than 70 leading ready-mix plants throughout the United States alone.

To facilitate selective distribution, **separate prints** of this title (42-34) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946.

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Mechanical Dispensing Devices for Air-Entraining Agents*

By E. M. BRICKETT†

SYNOPSIS

Several devices for accurate measurement of liquid admixtures as introduced into the concrete batch at the mixer are described and illustrated, with special reference to air-entraining agents as used in ready-mix and concrete products plants and on paving jobs. Since the quantity of the solution is relatively small, accuracy is important for uniform results.

In the production of air-entraining concrete by the introduction of air-entraining agent at the mixer, a metering device is essential to the accuracy and simplicity of the operation.

We have had some interesting experiences in the design and operation of several metering devices in the introduction of air-entraining agents in products plants, ready-mix plants, and on paving jobs.

The quantity of solution to be added to a batch is relatively small. The volume required ranges from one to one and one-half fluid ounces of "agent" per bag of cement. There are occasions when the required quantity is only a fraction of an ounce, and others when the figure is greater than two ounces per bag, but, in general, the figures of one to one and one-half ounces apply.

The simplest method of adding the air-entraining agent is, of course, by means of a dipper. A small tin can, cut down to the required capacity, and fitted with a handle, makes a very convenient tool for measuring and applying the agent. A pail of the solution is placed in a handy location and the agent is dipped from the pail into the batch in the weigh hopper or directly into the mixer.

A very simple measuring device which now is used in some products plants and ready-mixed concrete plants is shown in Fig. 1.

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Dewey & Almy Chemical Co., Cambridge, Mass.

In this particular meter a measuring glass is located between two cocks. The supply comes in from the left and with the right cock closed the supply is admitted through the left cock until the desired quantity shows in the calibrated glass. With both cocks closed the agent is held at the meter until required. Thus, by opening the right cock the agent is discharged from the meter to the batch.

In Fig. 2 is shown a proportioner developed particularly for use in ready-mixed concrete plants, though its application is not limited to such plants. This meter is filled and discharged through a three-way cock. In operation, the cock is set to the filling position and the meter is completely filled, the liquid finally filling the vent tube to the level of the liquid in the supply reservoir. A section of lucite tubing in the vent permits the operator to know when the meter is filled. Then, by turning the three-way valve a quarter turn to the discharge position, the liquid flows from the meter to the concrete batch. The amount discharged is governed by the setting of the calibrated quadrant. The discharge is rapid and has a sufficiently sharp cut off. The three-way valve prevents any possible discharge of liquid direct from the reservoir to the batch.

This meter is simple and inexpensive to build. There is no machine work required in its construction; it can be built in a welding shop.

Fig. 3 shows such a meter installed in a ready-mixed concrete plant. The meter is small and lends itself easily to installation in a convenient position. In this illustration the meter is mounted on the scale-beam housing where it is handy for the operator. In this particular installation the liquid is discharged into the sand in the weigh hopper. There are other installations in which the liquid agent is discharged into the water line and carried into the batch with the gauging water.

In ready-mix plants where the volume and proportions of successive batches are continually changing, it is a simple matter to set the meter for any desired quantity of air-entraining agent. The meter fills and discharges a measured quantity merely by a quarter turn of the three-way valve; the operator does not have to stand by to close the cock at a given moment.

In Fig. 4 is shown a meter which has been used on paving mixers to meter the air-entraining agent into the batches. This meter was designed and built by our Seattle distributor, the Charles R. Watts Co. It is entirely automatic in its action and insures a measured quantity of agent in every single batch. This meter is in effect a pump with an adjustable stroke. As the aggregate skip of the paver is raised it bears on the bar at the left and swings it in a counter-clockwise direction. Through the connecting link and the bell crank this forces down the plunger of the pump, forcing the liquid agent through the rubber hose to the water line

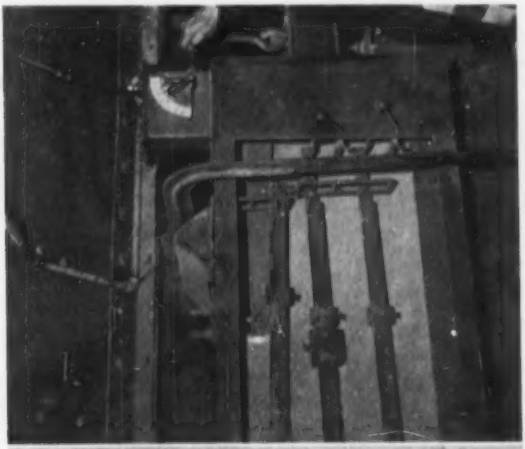

Fig. 3.—Proportioner installed in ready-mix plant.

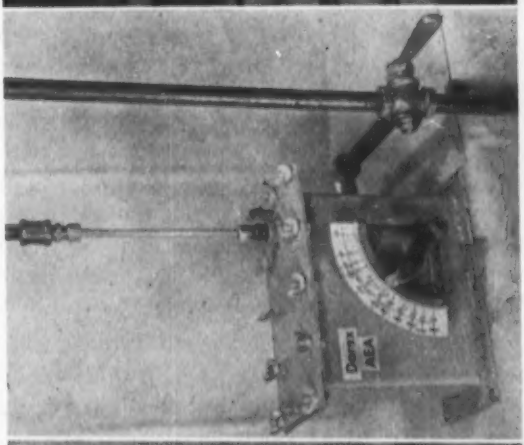

Fig. 2.—Brickett proportioner for ready-mix concrete plants.

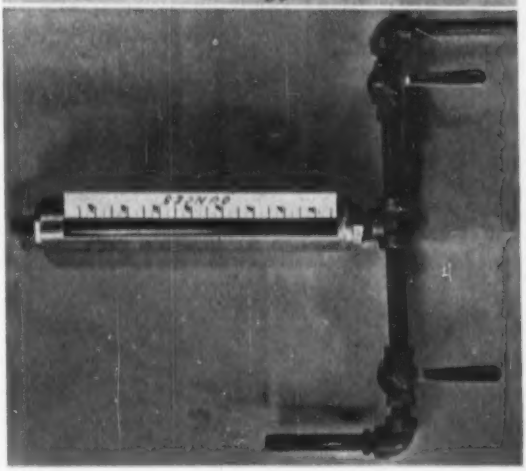

Fig. 1.—Simple measuring device.

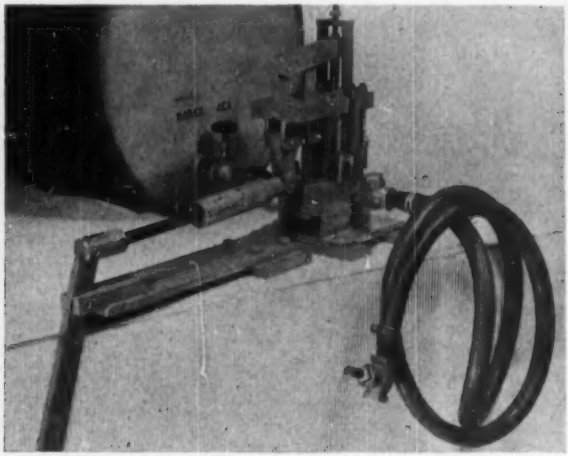

Fig. 4.—Watts meter.

between the water tank and the mixer drum. As the skip is again lowered the pump plunger returns to the adjustable stop, drawing in a new charge of agent in preparation for the next batch. The position of this stop, which is merely a set screw with a lock nut, determines the length of stroke and, consequently, the quantity of agent pumped to the batch. To adjust for any desired stroke the mixer skip is raised and the pump plunger is forced to the bottom of its stroke. While it is held there the set screw is adjusted with the aid of the proper one of three spacers shown attached to the pump by chains.

At one end of the tension link there is a spring over-run mechanism. The pump plunger reaches the end of its stroke while the skip still has a foot or more to go. The remaining travel of the skip is taken up in the over-run mechanism. With this arrangement the pump attains its full stroke even though the skip may not be completely raised. Also the skip may be rapped or vibrated without pumping more agent into the batch.

This meter is entirely automatic. It operates on the pressure principle and its performance is constant, even though the paver may be on a grade.

To facilitate selective distribution, **separate prints** of this title (42-35) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946.

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

A Simple Accurate Method for Determining Entrained Air in Fresh Concrete*

By S. W. BENHAM†

INTRODUCTION

This method, which is used by State Highway Commission of Indiana both in the laboratory and in the field, is based on the equation

$$\text{percentage of air} = \frac{(T\text{-}A)\ 100}{T} \text{ in which}$$

T = unit weight of the air-free concrete, and
A = unit weight of the concrete containing air.

The above equation is, of course, applicable regardless of the method employed for determining T and A. The principle involved in the experimental method described here, i.e., measuring volume by displaced water, is so old that no originality is claimed.

The determination of A is made by the same procedure as that employed in the yield test (cement content) using a 0.5 cu. ft. cast aluminum yield bucket.

T is determined by measuring the volume, by displacement in water, of a sample of fresh concrete of known weight. The apparatus used consists of the yield bucket and a hook gage which converts the device into a pycnometer the volume of which is calibrated with water of known temperature (0.452 cu. ft. in Table 1). The necessary data and computations for the complete air determination test consists of four weighings, five subtractions, three divisions, and one multiplication, all of which are extremely simple. See example in column 1 of Table 1.

PROCEDURE

After A is determined the operator removes concrete from the yield bucket until approximately 30 pounds remain (line E Table 1). After

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Indiana State Highway Commission, Indianapolis, Ind.

TABLE 1—FORM USED BY INSPECTOR IN THE FIELD

I.T. 595B

STATE HIGHWAY COMMISSION OF INDIANA

AIR CONTENT REPORT

CONTRACT NO._____ PROJECT NO._____ SECTION_____ 194____

Test Number	1	2	3	
Weight of Container and ½ cu. ft. Concrete	85.00			
Weight of Container (empty, clean, and dry)	10.00			
Weight of ½ cu. ft. Concrete	75.00			
Weight of 1 cu. ft. Concrete	150.00			A
Weight of Container and Concrete Sample	40.00			D
Weight of Container (empty, clean, and dry)	10.00			
Weight of Concrete Sample	30.00			E
Weight of Container, Concrete Sample and Water to Gauge Point	56.10			
Weight of Container and Concrete Sample	40.00			D
Weight of Water to fill Container to Gauge Point	16.10			F
Volume of Water in cu. ft. $\frac{F}{62.30}$	0.258			G
Calibrated Volume of Container to the Hook Gauge Point	0.452			V
Less Volume of Water—cu. ft.	0.258			G
Absolute Volume of Concrete Sample in cu. ft.	0.194			H
Weight of Solid Concrete on Air Free Basis Lbs. per cu. ft. $\frac{E}{H}$	154.64			T
Air Content $= \frac{T - A}{T} \times 100$	3.00			P

CALCULATIONS:—

_____

_____

Title

weighing (line D), sufficient water is added to inundate the sample completely. The hook gage is set in position and water added until the "dimple" in the water surface breaks. The hook gage is then removed and the gross weight obtained. From this line D is subtracted to obtain line F, the weight of water to fill the pycnometer to the hook gage point. This weight of water is converted into cubic feet, recorded in line G, and subtracted from the calibrated volume of the pycnometer (line V) to obtain line H, the absolute volume of the concrete sample in cubic feet. T is then obtained by dividing the weight of the sample (line E) by its absolute volume (line H) obtaining, as shown in the attached example, 154.64 lbs. per cu. ft. The formula, $\frac{(T\text{-}A)\ 100}{T}$, is then used to compute percentage of air which is recorded in line P.

ADVANTAGES OF THE METHOD

This method for determining T experimentally is considered to be much more satisfactory than the method in which T is arrived at by computation, by dividing the weight of the batch by the sum of the absolute volumes of all the constituent materials. In the latter method the batch weight of each of the five constituents (two sizes of coarse aggregate, sand, cement and water) is divided by its specific gravity and by the unit weight of water to obtain its absolute volume. All five absolute volumes are then added to obtain the absolute volume of the batch. The weight of the batch is then divided by the volume to obtain the theoretical air-free unit weight of the fresh concrete. Such computations are laborious and the computed value of T is of no reliability whatever unless corrections are made for the moisture content of each of the three aggregates. These corrections require additional testing at the batching plant and make the computations still further complicated. An additional criticism is that specific gravity of aggregate from a given source is sometimes variable. A still additional criticism is that a change in the batch weights requires a new computation.

In the experimental method the operator is not concerned with batch weights, specific gravities, nor moisture contents of the aggregates. If, unknown to the inspector, steel punchings were substituted for one of the sizes of coarse aggregate the validity of the method would be entirely unaffected. Such a substitution would, of course, be apparent at once because of the high values of both A and T. What the inspector is actually doing is determining the specific gravity of a sample of fresh concrete although he does not compute it because he wants to know unit weight only.

Both A and T are determined at the mixer thereby avoiding all travel back and forth between the batching plant and mixer.

Three independent tests, as provided for in Table 1, can be conducted and all computations performed within approximately a half hour. In addition to being quick the method and computations are so simple that the average inspector can learn to conduct the test satisfactorily with very little instruction. The fact that he is not concerned with the quantities nor properties of any of the materials in the batch makes his work extremely simple in comparison with the "sum-of-the-absolute-volumes" method.

CRITICISMS OF THE METHOD

It has been found that with excessive quantities of some foaming agents, sufficient to cause more than 7 per cent air, the removal of *all* the air by stirring becomes quite difficult. For that reason the method might be improved by the use of a water tight lid on the yield bucket so the apparatus can be rolled. However, this change in procedure, which would require additional equipment and more time, has not been considered necessary because our present specifications permit not more than 5 per cent air. The foam on the surface of the water, of course, obscures the hook gage point. Such foam can be dispersed instantly by one or two drops of a normal solution of amyl alcohol.

To facilitate selective distribution, **separate prints** of this title (42-36) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946.

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Effect of Use of Blended Cements and Vinsol Resin-Treated Cements on Durability of Concrete*

By W. F. KELLERMANN†
Member American Concrete Institute

SYNOPSIS

Presents a part of the results obtained from an investigation of the durability of concrete, by the Public Roads Administration using blends of portland cements with natural cements (86 per cent and 14 per cent by weight, respectively) and Vinsol resin-treated cements. Results presented in this contribution have a bearing on resistance to freezing and thawing tests, especially because of unusual results of a prolonged interruption of the daily freezing and thawing cycle.

The three methods now commonly employed for securing air entrainment in concrete are: (1) the use of air-entraining portland cements, (2) air-entraining admixtures added at the mixer and (3) the use of plain portland cements blended with blending cements containing an air-entraining agent. In the latter case the general practice has been to use a blend consisting of 1 bag of natural cement and 5 or 6 bags of plain portland cement, the air-entraining agent being carried by the natural cement.

This discussion concerns results of freezing and thawing tests obtained in connection with an investigation to determine the effect of the use of blended cements and Vinsol resin-treated cements on various properties of concrete. A paper giving complete results of this investigation is now in preparation and will be available at a later date. The results presented herein were selected because they bear directly on the subject of durability from the standpoint of resistance to freezing and thawing and because of the rather unusual results obtained in the freezing and thawing test resulting from a prolonged interruption in the daily freezing and thawing cycle.

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Senior Materials Engineer, Division of Physical Research, Public Roads Adm., Washington, D. C.

Included in the investigation were two plain portland cements; the same two cements interground with unneutralized Vinsol resin; and the two plain cements blended with six different blending cements in the proportion of 86 per cent portland cement and 14 per cent blending cement by weight. Five of the six blending cements were natural cements and of the five, two were from the same mill. One of these contained an air-entraining agent while the other did not. Aggregates were a river sand and a river gravel of 1-in. maximum size. The mix used with the plain cements contained approximately 5¾ bags of cement per cu. yd. and 6 gal. of water per bag. For those mixes containing the treated cements or the blends, the same weight proportions were used as were used with the plain cements. However, the water was varied so as to keep the slump within the range of 3 to 4 in. for all mixes.

Test specimens were 3- by 4- by 16-in. beams which were alternately frozen and thawed in water, the temperature range being from 70 to -20 F, one cycle being made per working day. The natural frequency of each specimen was determined dynamically just before freezing and thawing started and after the completion of each five cycles of freezing and thawing. Readings were taken on the 3-in. and also on the 4-in. faces of each specimen, there being two specimens for each variable investigated.

Two separate series of freezing and thawing tests will be discussed. In the first series the specimens were given an initial curing of 230 days in the moist room at 70 F. The results of this series are given in Fig. 1, which shows the relation between freezing and thawing cycles and the reduction in the natural frequency squared (N^2), which is a measure of the reduction in the dynamic modulus of elasticity. In the left panel results are shown for cement A plain, cement A treated, and cement A plain blended with the six blending cements. Reference to the panel indicates a marked difference in behavior for the different combinations tested. For instance, specimens representing the plain cement A (1) and the blends A (1) + II (a) and A (1) + VI broke down very rapidly. The air contents of these three combinations were all of the order of 1½ per cent. Somewhat better resistance was shown by the blend A (1) + I, which had an air content of the order of 2 per cent. The remaining combinations, treated cement A (2) and the blends A (1) + II (b), A (1) + III and A (1) + V, all showed superior resistance. The range in air content for this group was of the order of 3 to 5 per cent. It is of interest to note that blending cement II (b), included in one of the combinations in the latter group, was the same product as blending cement II (a) except that it carried air-entraining material. This was reflected in the air contents of the two different combinations and serves as an ex-

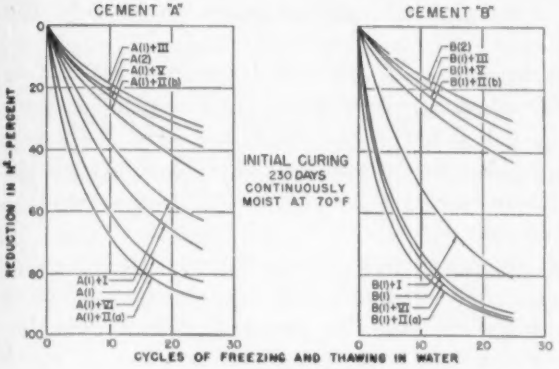

Fig. 1—Effect of use of blended cements and Vinsol resin-treated cements on resistance to freezing and thawing (First series).

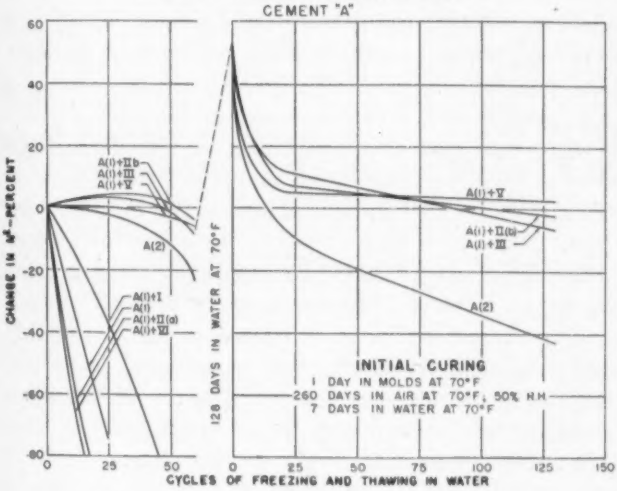

Fig. 2—Effect of use of blended cements and Vinsol resin-treated cements on resistance to freezing and thawing (Second series, Cement A).

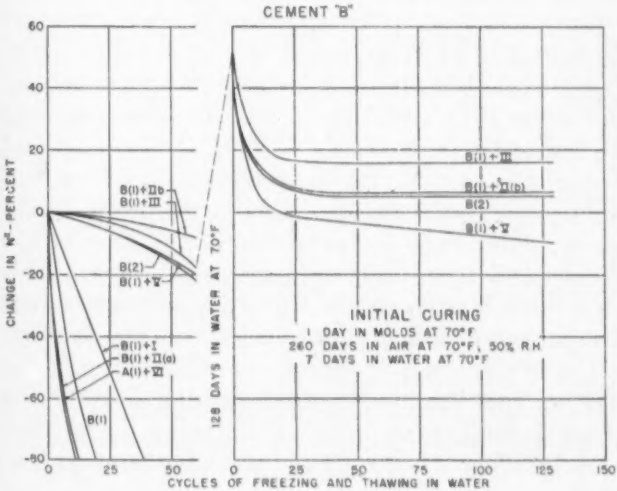

Fig. 3—Effect of use of blended cements and Vinsol resin-treated cements on resistance to freezing and thawing (Second series, Cement B).

planation for the difference in behavior between blends A (1) + II (a) and A (1) + II (b).

Results obtained with cement B are shown in the right panel of Fig. 1. Although somewhat poorer resistance was shown for the plain cement B (1) and the blends B (1) + I, B (1) + II (a) and B (1) + VI than in the case of cement A, the order of resistance for the eight combinations was, with one minor exception, exactly the same as shown for cement A. The general trends were also the same.

Because of the great difference in results shown between the different combinations after 25 cycles of freezing and thawing, it was decided to test all specimens in flexure and compare the results with tests on companion control specimens which had been stored continuously in moist air at 70 F. This was done with the results shown in Table 1. The table indicates good correlation between the reduction in modulus of rupture and the reduction in the natural frequency squared.

The second series of freezing and thawing tests was conducted with companion specimens which had been originally used for volume change measurements. Specimens were cured in the molds under wet burlap for 24 hours and then stored in air at 70 F. and at a relative humidity of 50 per cent for 260 days. They were then immersed in water for 7 days before freezing and thawing started. After 60 cycles of freezing and thawing it was necessary to interrupt the daily cycle because of the urgent need of the freezing equipment for work related to the war effort. These first 60 cycles of freezing and thawing will be referred to as the first phase. Accordingly, the specimens were stored in water for 128 days after which the daily cycle was again started and continued for 130 additional cycles. This will be referred to as the second phase.

Results of these interrupted tests are shown in Fig. 2 and 3. Fig. 2 gives results for cement A and from it the same trends during the first 60 cycles of freezing and thawing (first phase) are exhibited as were shown in Fig. 1. A very rapid breakdown is indicated for plain cement A (1) and the blends A (1) + II (a) and A (1) + VI. The group showing good resistance in the first series, A (2), A (1) + II (b), A (1) + III and A (1) + V, also showed the best resistance in this series. However, combination A (2) gave poorer resistance than any of the blends in the good group. Of interest is the shape of the curves for the good group, which show an opposite trend from their companions in Fig. 1. This is due to the fact that there was no loss in frequency during the first few cycles. In fact some combinations showed a gain during the first 40 or 50 cycles. All specimens gained slightly in weight during the first few cycles, indicating additional curing after freezing and thawing started. Apparently the 7-day immersion period following the 260-day air curing treatment was insufficient to completely saturate the specimens.

At the completion of 60 cycles of freezing and thawing, the shape of the curves for three combinations in the good group indicated a rapid rate of deterioration, particularly combination A (2).

As previously explained, the test was interrupted at this time and the specimens from the good group were stored in water at 70 F for 128 days, after which the freezing and thawing cycle was resumed. Frequency readings taken at the end of the water storage period indicated an increase of 40-50 per cent in N^2 over the zero readings taken at the time freezing and thawing was first started (beginning of the first phase). For the first 15 cycles of freezing and thawing immediately following, the percentage reduction in N^2 was at a very rapid rate. However, at 20 cycles the curves, with the exception of the one representing combination A (2), flattened out to the extent that at the end of 130 cycles of freezing and thawing there was little change in N^2 from the original zero taken at the beginning of the first phase. In the case of combina-

TABLE 1—EFFECT OF THE USE OF BLENDING CEMENTS AND VINSOL RESIN TREATED CEMENTS ON PROPERTIES OF CONCRETE

Results of freezing and thawing tests
First Series

Reduction in modulus of rupture and frequency squared after 25 cycles of freezing and thawing in water.

Portland cement	Blending cement	Reduction in	
		Modulus of rupture	N^2
		Percent	Percent
A (1)	None	69	72
A (2)	None	32	35
A (1)	I	61	66
A (1)	II (a)	82	88
A (1)	II (b)	57	48
A (1)	III	40	33
A (1)	V	45	40
A (1)	VI	77	82
B (1)	None	86	92
B (2)	None	37	32
B (1)	I	77	81
B (1)	II (a)	88	94
B (1)	II (b)	57	44
B (1)	III	48	35
B (1)	V	47*	39
B (1)	VI	89	93

*One control test only.

Specimens 3 x 4 x 16-in. beams stored continuously in moist air 230 days prior to start of freezing test. Each result average of two tests unless otherwise noted.

Reduction in modulus of rupture based on strength of control specimens stored continuously in moist air.

tion A (2) the decrement was about 40 per cent. Based on results obtained in the first phase, it would be expected that combination A (2) would not stand up as well as the other combinations tested in the second phase. Apparently due to the greater decrement at the end of the first phase (24 per cent), the rest period of 128 days in water was not as effective in producing the same degree of resistance for this combination as was obtained with the other three combinations.

It is worthy of note that the shape of the curves in the second phase is not the same as those representing the same combinations in the first phase but are similar in shape to those shown in Fig. 1. The data shown in Fig. 1 and for the second phase in Fig. 2 were obtained with specimens which had been subjected to prolonged moist curing prior to the start of freezing and thawing. On the other hand, the data shown for the first phase in Fig. 2 were obtained with specimens which had been subjected to prolonged air curing followed by but 7 days of moist curing prior to freezing and thawing. The above discussion is presented as an explanation of the difference in the general shapes of the curves between the first and second phases and not as an explanation of the general nature of the results shown for the second phase.

The fact that specimens which have been subjected to cycles of freezing and thawing will, when given a rest period in water, show a recovery in dynamic modulus of elasticity, has been reported by other investigators.* However, in the tests referred to, the specimens were tested for strength after a water storage period of 30 days, so that data similar to that shown in the right panel of Fig. 2 were not obtained. The data herein reported indicate, in three cases out of four, that there was practically no decrement in dynamic modulus due to the combined 190 cycles of freezing and thawing to which the specimens were subjected. Compared with the data shown in the left panel of Fig. 1 for the same combination, a much greater decrement would be expected.

It is well known that the moisture condition of the specimens at the start of the test has an influence on freezing and thawing results, thoroughly saturated specimens showing poorer resistance than partially dried specimens. This would explain the superior resistance shown in the first phase of the second series (left panel of Fig. 2), particularly for those combinations of relatively high air contents. On the other hand the excellent resistance shown in the second phase (right panel of Fig. 2) was obtained with specimens that had 128 days of water curing in addition to the water curing available during the first phase of 60 cycles of freezing and thawing. It would appear, therefore, that the results obtained in the second phase were influenced by the 128-day rest period,

*Proc. of the Highway Research Board, v. 24, 1944, p. 196, fig. 17.

during which time the specimens had an opportunity for recovery because of favorable curing conditions.

Results obtained with cement B are shown in Fig. 3. The trends follow those shown for cement A in Fig. 2 and for that reason no detailed discussion of the results will be presented.

It is believed that the results shown in Fig. 2 and 3 for the second phase of freezing and thawing (right panels) explain some of the erratic results that have been reported in the past. Interrupting the daily cycle for short periods and allowing the test specimens to remain unfrozen, may result in a recovery in dynamic modulus. For that reason, it is recommended that where interruptions are necessary because of non-work days, the test specimens be kept in a frozen rather than in a thawed condition.

The results discussed in this paper are presented at this time because of the trends shown in Fig. 2 and 3 resulting from the interruption of the daily freezing and thawing cycle. The completed report will include strength and volume change data in addition to data from a third series of freezing and thawing tests involving tests on discs cut from cores which were frozen and thawed in a calcium chloride solution.

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT, MICHIGAN **June 1946**

Air-Entraining Concrete—Pennsylvania Department of Highways*

By W. H. HERMAN†

SYNOPSIS

The experiences of the Pennsylvania Department of Highways with air-entraining concrete in which 331,555 bbl. of normal strength portland cement containing Vinsol resin were used since 1940, are reported. The Pennsylvania department's attitude on the subject of air entrainment is characterized by more concern with the particular admixture used than with the percentage of air entrainment and such use was inspired by difficulties with finer ground cements which prompted seeking an additive to improve pavement durability.

The experience of the Pennsylvania Department of Highways with air-entraining concrete dates from 1940. Since that time we have used 331,555 bbl. of normal strength portland cement containing Vinsol resin. We do not have voluminous laboratory or field data covering the development or effects of specific quantities of entrained air.

Pennsylvania's concrete suffered considerably with the cement industry's development of finer ground cements which were to produce greater plasticity or workability accompanied by higher and earlier strengths. We, therefore, sought an additive to improve the durability of our concrete.

Two experimental projects were constructed in 1940 using normal strength portland cements, of fine and coarse grinds, and the same cements to which pulverized Vinsol resin and tallow were interground with the clinkers at the rate of 0.18 lb. per bbl. of cement. The purpose of those additives at that time however, was to determine their effectiveness as deterrents of concrete pavement surface deterioration. The weight reduction or air entrainment of the cement containing the additives was

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
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determined as a matter of record and for future reference, and was based on unit weights of the various concretes.

Our views concerning the exploitation of the air entrainment feature of cement may provoke considerable criticism when we say that we are more interested in the additive causing the air entrainment than we are in obtaining a specific percentage of air entrainment or in recording the air entrainment on a percentage basis as is now generally promoted. We have accepted Vinsol resin as an additive to normal strength portland cement as a deterrent. On the basis of the first two experimental projects, we dismissed tallow as an alternate additive because of the difficulty of handling the cement itself; because of its tendency to cause surface foaming of the concrete and because of the inconsistent workability of the concrete. We have never used Darex because none of the 34 plants furnishing cement to our Department used or promoted the product.

Our first Vinsol resin experimental project (Route 84, Section 10, Crawford County, constructed 1940) is in an area having many freeze-thaw cycles. The concrete mix was not designed on an absolute solid volume basis. The results to date are very gratifying. Our second Vinsol resin experimental project (Route 271, Section 4, Crawford County, constructed in 1942) is in the same area. The concrete mix was designed on the absolute solid volume basis and controlled by means of the slump test. No provision was made on the first project for a reduction of fine aggregate but in the designed concrete of the second project it was found necessary to vary the fine aggregate volume for Vinsol resin concrete.

A third Vinsol resin project (Route 67054, Sections 1A and 2A, City of Philadelphia, constructed in 1942-43) started in the fall and completed the following spring was authorized not as an experimental project. No decrease was made in the fine aggregate of the concrete mix as no thought was given at that time to the air-entrainment control feature. The district engineer took exception to the slow initial set and the low strength of the concrete. We observed the lower flexural and compressive strengths obtained on our experimental projects and saw no cause for alarm. When paving operations were continued in the spring of 1943, we recognized the Portland Cement Association design adjustment of the fine aggregate to compensate for the entrained air. Being reluctant to reduce the mortar content, we cut approximately 3 per cent of the total fine and coarse aggregate. The necessary workability and weight loss were uniformly obtained.

During 1943 and 1944, three other Vinsol resin sections of the same route were constructed. Three different brands of cement were used but the same design and the same fine and coarse aggregates were used with favorable results.

In 1944 a 45-ft. single span reinforced concrete bridge (Route 103, Section 6, Tioga County) was constructed with Vinsol resin cement. The sand was extremely coarse and the crushed gravel coarse aggregate was generally flat and elongated. Here we were unable to develop air entrainment with a reduced mortar content and again found it necessary to reduce the total fine and coarse aggregate as we did in Philadelphia. Satisfactory weight losses were obtained. The flexural strength, although lower than regular concrete, was higher than anticipated. This project demonstrated the cohesive factor of the Vinsol resin because normally, flat and elongated aggregate requires excessive mortar to hold the particles. We also observed the lack of mobility in placing the concrete in structures. Chutes were replaced with elephant trunks so that the concrete was placed at the required location instead of flowing the mass concrete. There was no segregation in any part of the structure. Internal vibration was tried and found to be of no practical value, because the cohesive properties of the Vinsol resin reduced its effectiveness. External vibration was not tried.

A third experimental pavement project (Route 238, Section 14 and Route 320, Section 2, Mercer County, constructed in 1944) contained alternate sections of Vinsol resin cement, slag puzzolan cement and normal strength portland cement manufactured by the same company, but the normal cement was from different plants. The coarse aggregate was slag and, despite sand reduction, we were unable to obtain a weight loss in the Vinsol resin concrete.

In 1945 we used 194,956 bbl. of Vinsol resin cement in various pavement and structure projects scattered throughout the state. Our concrete designs are formulated at the sight of the project by laboratory-trained, field materials engineers. Up to this time, we could not develop air entrainment in hand mixes. Then the Hercules Powder Co. produced Vinsol resin in a neutralized form (sodium resinate), first added to the cement in liquid form and now in powdered form (Vinsol NVX). The improved product provided increased lubrication. The water content was reduced slightly and entrained air is injected into our hand mixes.

We learned the reason we could not obtain the weight loss on slag concrete was because of the differences of specific gravities of the minus 1½-in. and plus 1½-in. slag coarse aggregate which are proportioned separately, and also due to variations of the specific gravities during production of the slag. The same condition occurs to a slight extent with gravel. This means we may have to make specific gravity checks at the sight of the work.

We have developed a method for determining pavement deterioration, progressively classified with each type having a successively higher numerical value. These values are applied to each pavement slab

during a condition survey. The values are then factored to produce numerically a type rating for the entire pavement with zero being a perfect surface.

With reference to our Vinsol resin experimental projects we quote deterioration ratings (Table 1) based on annual condition surveys for 1943, 1944 and 1945.

TABLE 1—DETERIORATION CLASSIFICATION

	1943	1944	1945
Rt. 85, Section 10, Crawford County constructed 1940			
A-1 S.S.A. 1750-2000	16.20	18.82	22.99
A-2 S.S.A 1750-2000 +Vinsol resin	.59	1.78	2.16
B-1 S.S.A 1275-1450	11.74	17.58	19.83
B-2 S.S.A 1275-1450 +Vinsol resin	.15	1.58	2.45
Rt. 271, Section 4, Crawford County constructed 1942			
A-1 S.S.A. 1750-2000	7.77	10.09	12.01
A-2 S.S.A. 1750-2000 +Vinsol resin	0.00	0.02	0.25
B-1 S.S.A 1275-1450	0.11	0.26	0.94
B-2 S.S.A 1275-1450 +Vinsol resin	0.00	0.01	0.38
Route 238, Section 14 and Rt. 320, Section 2 Mercer County constructed 1944		1946	
Normal Strength, Plant A		1.76	
Normal Strength, Plant B		2.50	
Slag—Puzzolan, Plant B		0.82	
Normal Strength + Vinsol resin, Plant B		0.39	

The results in every project where Vinsol resin was used show remarkable improvement in resistance to deterioration.

Sometime ago we decided to learn more about our first Vinsol resin experiment so we determined the air content of the cements with the following results:

Cement	Per cent Air*
A-1 Normal Strength S.S.A.—1750-2000	5.7
A-2 Normal Strength S.S.A—1750-2000 + Vinsol resin	14.8
B-1 Normal Strength S.S.A—1275-1450	9.7
B-2 Normal Strength S.S.A.—1275-1450 + Vinsol resin	16.0

Plotting the turbidimeter analyses from 60 micron to 7.5 micron, we found that although A-1 cement had 23.82 per cent of 7.5 micron against 13.80 per cent of 7.5 micron for B-1, the percentages of the successive coarse sizes of the gradations paralleled each other. From the pavement results and the entrained air results of B-2 cement the question arises whether a slightly higher air content of B-1 would eliminate the necessity of an additive.

Cores were cut from the third experimental project approximately three months after the project was completed. The compressive strength

*A.S.T.M. C 185.

of the Vinsol resin cores was higher than with other cements. One core of each set was sawed in three approximately equal parts and permanently designated as top, center and bottom. From these we learned that the absorption of the top part of the cut cores for all types of cement was much lower than the center and bottom parts. This appears to indicate a lack of compaction. However, the comparative specific gravity of the different parts of the cut cores show fluctuations but not to the same marked degree.

Results of the freezing and thawing tests in a ten per cent solution of calcium chloride at -10 F. and thawed at room temperature developed the following results.

The normal strength portland cement concrete (Plant A) started to disintegrate at 30 cycles and was completely disintegrated at 55 cycles.

From Plant B, the concrete started to disintegrate at 40 cycles and was completely disintegrated at 90 cycles.

The slag puzzolan concrete started to disintegrate at 40 cycles and was completely disintegrated at 90 cycles.

The Vinsol resin concrete started to disintegrate at 85 cycles and at the completion of 100 cycles, 64 per cent of the concrete was still intact.

We have not investigated the efficiency of minus 50 and minus 100 mesh fine aggregate particles in the development of entrained air. Our specification for concrete design is based on the maximum solid volume of the aggregate with a maximum water content per bag of cement. The specification is broad and of necessity, permits variables to handle problems involved in using 34 brands of cement, 91 sources of sand, 222 sources of stone, 16 sources of slag and 63 sources of gravel. The design specification has been in effect for 5 years and most of the problems concerning variation in character and compatibility of material have been solved. From 1940 to 1944 inclusive, our field engineers made a total of 3446 concrete designs of which 691 had to be redesigned while in use. Vinsol resin, as an air-entraining additive, helped considerably to overcome the flat and elongated coarse aggregate problem as well as some of the fine aggregate problems, and added considerably to the workability of poor aggregate combinations. Our field men have been given directions to be followed in developing entrained air to obtain a weight loss of from 3 to 6 lb. per cu. ft., assuming normal strength portland cement concrete to have one per cent of entrained air recognized to be nonuniformly distributed. They are privileged, however, to work emperically, based on their experience with the aggregates in their sections, to obtain a practical design with the lowest water content applicable to placing of the concrete. In doing this, they will reduce the fine aggregate only or both the fine and coarse aggregate. In com-

puting the yield, a nominal figure of 0.25 gal. of water is included as the additional amount required for non air-entrained cement.

We have tabulated the average weight loss and average per cent of air entrainment covering all designs made during 1945. (Table 2).

TABLE 2

	No. of Designs	1:2:3.50	No. of Designs	1:2:3.75	No. of Designs	1:2.5:4.5
Average Wt.—Loss Pounds						
Stone	13	4.1	14	4.3	5	4.2
Gravel	12	3.7	17	3.6	11	4.0
Slag	3	4.5	4	4.5	1	5.5
Average Weight—Loss Percent						
Stone	13	2.6	14	2.8	5	2.8
Gravel	12	2.2	19	2.3	11	2.6
Slag	3	3.2	4	3.2	1	4.1
Water Gallons —Per Bag						
Stone	16	5.08	18	5.36	8	6.41
Gravel	16	4.71	20	5.06	15	5.98
Slag	3	5.18	7	5.50	1	6.55

Personnel not being available for several years we have not studied the issue of optimum or maximum percentage of air entrainment of the concrete or of the ratio of the air entrainment of the concrete to that of the cement. At present we see no advantage in designing for a specific per cent of air entrainment of the concrete. We are, therefore, requiring a 3 to 6 pound weight loss in our revised specifications. We are in accord with specifying 16 per cent ± 4 per cent for the air entrainment of the cement rather than a percentage of Vinsol resin but, as yet, see no advantage.

To facilitate selective distribution, **separate prints** of this title (42-38) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946.

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Portland-Rosendale Cement Blends Give High Frost Resistance*

By B. H. WAIT†
Member American Concrete Institute

SYNOPSIS

Results are reported on numerous paving jobs in northeastern states, in the support of the use of portland cement blends as a means of reducing disintegration from frost action where the air entrained averaged about 1 per cent only. Results were satisfactory and the weight of the concrete was higher than for straight portland-Vinsol resin mixes.

This discussion is from the viewpoint of the manufacturers and consumers of cementing materials which, when blended with portland cement, produce the desired results uniformly in resistance to freezing and thawing and at the same time assist in the production of a more desirable concrete from all other angles.

In the northeastern section of the United States the portland-Rosendale blend over a period of ten years has produced very satisfactory results. These results in their relation to freezing and thawing have been aided by a very small amount of grinding compound or waterproofing. The amount of entrained air in the millions of square yards of roads constructed, has shown such a small loss in weight that this work should not be classified as "air-entrained concrete." The extra air entrainment in excess of that in the standard untreated portland cement concrete has averaged about 1 per cent.

On these roads which have been built under ordinary construction conditions with ordinary materials, the amount of air entrainment and the resistance to freezing and thawing has not been affected by the gradation or quantity of fine aggregate used or by any other conditions of standard construction with the exception that very high water content used in transit mix trucks has caused extra loss of weight in some cases in structural work.

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†The Wait Associates, Inc., New York, N. Y.

I present data from a few jobs which are well known and have been well publicized:

1) The Wellsville-Bolivar Road in Alleghany County was built partly with standard portland, the Rosendale-portland blend and the treated portland, about fifteen or twenty different cements being shipped in from a wide variety of mills throughout the U. S.

On that job the average weight per cu. ft. of the standard portland cement concrete was 149.4 lb.

The average weight of the portland Vinsol resin concrete was 145.5 lb.

The average weight of the Rosendale standard portland blend with the same basic materials was 148.3 lb.

The average loss on the Rosendale blend was 1.1 lb. as against 3.9 lb. for the Vinsol resin concrete.

The average length of life in freezing and thawing was 156.1 cycles for the blend, as against 119.3 cycles for the air-entrained portland-Vinsol.

These results, from a New York State Dept. Public Works report, indicate the greater resistance to freezing and thawing with only about 25 per cent of the air voids in the Vinsol treated cement. Four portland cements were used blended with the Rosendale. The concrete was laid at the ordinary 2-in. slump generally used in highway construction with the portland blend.

2) On the Storm King Highway in Orange County the following weights per cu. ft. are reported by the New York State Highway Dept. on samples of concrete two years old, with 6 or more portland cements used:

Average portland with 80 lb. Rosendale replacement	150.24 lb.
Average portland with 94 lbs. Rosendale replacement	151.56 lb.
Average Vinsol resin portland	146.25 lb.

The resistance to freezing and thawing was substantially the same with the different combinations, the loss on all of them being so low that very excellent results were to be expected.

However, in this connection results in freezing and thawing were obtained with 4-5 lb. less air entrainment and air voids in the blend as against the Vinsol resin.

3) On the Idlewild Airport in New York City additional data will shortly be available, but we are able to advise at this time that the amount of air entrainment in the portland-Rosendale blend is about 1 per cent and averages no greater than the air entrainment found in the standard portland mix. On this work the strengths—both compressive and flexural—have been extremely satisfactory. Freezing and thawing tests have not been carried to destruction but have been in the freezer

sufficiently long to indicate that the concrete will have extreme durability. The average loss at 100 cycles was about 4 per cent. The results are typical of millions of cubic yards of concrete laid under standard conditions and with the low water cement ratio applicable when the Rosendale blends are used.

A summation seems to indicate that with an improved cementing material little if any grinding aids or air voids are necessary for the protection of concrete under the worst practical conditions.

During the last two years we have experimented in the field and in the laboratory with cementing materials other than the Rosendale and the results with at least one other material available in large quantities and over a wide area give full protection against freezing and thawing, with a very small amount of grinding aid if that aid is of the right quality.

On one job with this new material freezing and thawing resistance has been practically perfect with high strength and other desirable qualities with as little as one third of .01 per cent of a special grinding compound or approximately one-eighth of the average amount used with portlands.

In view of past experience is it not desirable that attempts be made to produce a better and more uniform cementing material either by blending the portland with other cementitious materials or by manufacturing a better portland cement instead of attempting to protect unsatisfactory cements with waterproofing or grinding aids with all of the attendent non-uniformity in the concrete in the field? This has been done by the New York State Dept. of Public Works for 7 or 8 years with uniformly excellent results from all angles by combining the best portland cement obtainable with a satisfactory cementitious blend. It appears that while a small percentage of air-entraining agents of the right kind is desirable, it is not necessary to rely on them entirely for freezing and thawing resistance as is proposed in the air-entraining concrete.

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Discussion of a Symposium:*

Entrained Air in Concrete

By W. F. KELLERMANN, THOMAS E. STANTON, M. SPINDEL, W. A. CORDON and HENRY L. KENNEDY

By W. F. KELLERMANN†

[Editor's note: This discussion refers particularly to the paper by S. W. Benham: "A Simple Accurate Method for Determining Entrained Air in Fresh Concrete," p. 677.]

In order to compare results obtained with the hook gage method developed by Indiana with other procedures used for determining air content, a series of comparative tests was recently made in our laboratory, using both the hook gage and the Pearson Pycnometer‡. The data are shown in Table A, together with the air contents calculated in accordance with A.S.T.M. Method C 138-44.

The technique employed for eliminating the scum when using the Pearson pycnometer was somewhat different from that suggested by Mr. Pearson. Our technique may be described as follows: after the scum is floated into the tube of the pycnometer, it is dissipated by pouring a known quantity (usually 150 ml.) of alcohol into the tube. The alcohol dissipates the scum and most of it floats on top of the water in the tube, making it unnecessary to make a correction for the combined volumes of the alcohol and water as is ordinarily done when the two liquids are mixed. The alcohol used for this purpose consists of 95 per cent ethyl alcohol and 5 per cent methyl alcohol.

Using the procedure described above, we have had no difficulty in dissipating scum resulting from the use of air-entraining cements and different air-entraining admixtures. However, for air contents of the order of 8 to 10 per cent, it has been found necessary to use about 300 ml. of alcohol instead of the usual amount.

For air contents up to 3 per cent (gravimetric) the hook gage gave values about 0.8 percent higher than the gravimetric procedure and about

*ACI JOURNAL, June 1946, *Proceedings* V. 42 pp. 601.
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‡A.S.T.M. *Proceedings* V. 44, 1944, p. 343.

0.2 percent higher than the Pearson pycnometer. In the 3 to 6 percent range, the hook gage and pycnometer gave values about 0.1 to 0.4 percent higher, respectively, than the gravimetric procedure. For air contents above 6 percent, the hook gage results were all lower than those obtained by the gravimetric procedure, the greatest divergence being for the extremely high air content of 10.3 percent. In this connection, attention is directed to the fact that no attempt was made to remove all the air by placing a cover over the container and rolling in accordance with the procedure suggested by Mr. Benham for use in cases where the concrete contains an excessive amount of air.

TABLE A—COMPARATIVE TESTS FOR AIR CONTENT OF CONCRETE

Mix No.	Air in concrete, per cent			Variation from gravimetric percentage points	
	Gravimetric A.S.T.M. C138	Volumetric			
		Pearson pycnometer	Hook gage	Pearson pycnometer	Hook gage
	Rounded coarse aggregate, 1 inch maximum size Cement factor, 5.5 sacks per cu. yd. Slump 4½-6 in.				
1	0.5	1.2	1.6	+0.7	+1.1
2	0.7	1.2	1.9	+0.5	+1.2
3	0.4	1.2	1.4	+0.8	+1.0
4	0.6	1.3	1.3	+0.7	+0.7
5	0.7	1.3	1.7	+0.6	+1.0
6	0.6	1.3	2.3	+0.7	+1.7
7	0.5	1.3	1.5	+0.8	+1.0
8	0.7	1.3	1.7	+0.6	+1.0
9	0.5	1.3	1.4	+0.8	+0.9
10	0.7	1.3	1.3	+0.6	+0.6
11	0.6	1.3	1.3	+0.7	+0.7
12	0.9	1.5	1.6	+0.6	+0.7
13	0.9	1.5	1.3	+0.6	+0.4
14	0.7	1.6	1.6	+0.9	+0.9
15	0.9	1.6	2.2	+0.7	+1.3
16	1.1	1.7	2.1	+0.6	+1.0
17	1.2	1.7	1.8	+0.5	+0.6
18	1.0	1.9	1.7	+0.9	+0.7
19	1.1	1.9	3.0	+0.8	+1.9
20	1.7	2.2	2.5	+0.5	+0.8
21	2.1	2.3	2.4	+0.2	+0.3
22	2.8	3.1	3.9	+0.3	+1.1
23	4.0	4.6	3.9	+0.6	−0.1
24	6.0	6.2	6.0	+0.2	0
25	6.4	6.9	6.4	+0.5	0
26	7.2	7.8	6.8	+0.6	−0.4
27	10.3	10.7	8.6	+0.4	−1.7

TABLE A—CONT'D

Mix No.	Air in concrete, per cent			Variation from gravimetric percentage points	
	Gravimetric A.S.T.M. C138	Volumetric Pearson pycnometer	Volumetric Hook gage	Pearson pycnometer	Hook gage
	Angular coarse aggregate, 1-inch maximum size. Cement factor, 6.0 sacks per cu. yd. Slump 4-4½ in.				
28	2.4	3 5	2.2	+1.1	−0.2
29	3 6	3.9	3.9	+0.3	+0.3
30	3.9	4.1	3.8	+0.2	−0.1
31	3.9	4.4	4.2	+0.5	+0.3
32	4.0	4.2	4.3	+0.2	+0.3
33	7.5	7.3	7.3	−0.2	−0.2
34	9.1	9.2	8.4	+0.1	−0.7
	Rounded coarse aggregate, 1½-inch maximum size. Cement Factor, 6.0 sacks per cu. yd. Slump 4-6 in.				
35	2.9	3.5	3.1	+0.6	+0.2
36	5.1	5.6	5.4	+0.5	+0.3
37	6.2	6.6	6.1	+0.4	−0.1
	Angular coarse aggregate, 1½-inch maximum size. Cement Factor, 6.0 sacks per cu. yd. Slump 2½-3½ in.				
38	3.0	3.3	3.0	+0.3	0
39	5.3	5.6	5.4	+0.3	+0.1
	Rounded coarse aggregate, 2-inch maximum size. Cement Factor, 6.0 sacks per cu. yd. Slump 2 in.				
40	2.0	3.0	2.8	+1.0	+0.8
	Angular coarse aggregate, 2-inch maximum size. Cement Factor, 6.0 sacks per cu. yd. Slump 3 in.				
41	3.5	4.3	3.5	+0.8	0

Each value represents one determination.
Hook gage tests made with one-half cubic foot measure.
Mixes 1-21, incl., 21 different plain cements.
Mixes 22-27, incl., 6 different treated cements.
Mixes 28-41, incl., 1 plain cement with 7 different air-entraining admixtures added at mixer.

It is believed that a long-handled spading tool about 4 in. in width is a more suitable instrument for agitating the concrete for the purpose of removing air than the tamping rod. By using the spading tool in a twisting motion it is possible to dislodge large aggregate particles, such as 2 in. crushed stone, that tend to key in the bottom of the container, and to bring them and the adjacent mortar to the surface, where the entrained air is more readily released. The use of this size and type of aggregate has occasioned some difficulty in agitating the concrete. However, with a maximum size of 1½ in. or less, the same difficulty was not experienced.

By THOMAS E. STANTON*

Those papers in the Symposium on Entrained Air in Concrete which relate to the *merits* of air entrainment describe only tests to determine resistance to freezing and thawing and, except for the brief description

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by Mr. Meissner of some studies carried on by the Bureau of Reclamation, as published in the previous Symposium, V. 40, p. 522, ACI JOURNAL June, 1944, so far as this writer is aware, no description has been published of tests to determine the possible beneficial attributes of air entrainment in providing resistance to attack by the sulfates found in alkali soils.

The fact that air entrainment may be beneficial in this respect first came to the attention of the writer about three years ago.

About a year ago we found time to undertake a study of the benefits of air entrainment in concrete subjected to long time direct exposure to a high alkali content soil.

The cements chosen for the test were intended to be Types I and II of two brands. Actually one of the cements furnished as Type II contained 9 percent C_3A and was, therefore, not a true Type II cement (Table B). The cement content of the concrete mixtures was fixed at 4 and 5.5 sacks per cu. yd.

Two air entraining agents were used, one being Vinsol Resin.

The results with air entraining agent No. 1 were only slightly superior to the plain concrete, but a very marked improvement is noted in the case of air entraining agent No. 2 (V.R.).

The order of disintegration of the concrete has been in the order of the C_3A content of the cement. Only slight disintegration has been noted to date in the case of the lower C_3A cements.

After one year exposure the highest C_3A cement 4-sack concrete specimens have completely disintegrated even when V.R. was added. The same performance was noted in previous tests where the cement factor was low and the C_3A high (11 percent or more.) When the cement factor is increased to at least 5.5 sacks per cu. yd. air entrainment appears to become more effective, particularly as the C_3A content decreases.

Fig. A shows the condition of the alkali exposed specimens in round 1 of the current series of tests. There were four specimens of each cement and mix, therefore four rounds. The condition of the specimens in round 1 is typical of the condition in all four rounds.

While the real superiority of the air entrained concrete in the case of the lower C_3A cements is not apparent to date, a close inspection of all specimens in the test series and past experience indicates that within another year a very marked differential will become apparent.

TABLE B—PHYSICAL AND CHEMICAL TESTS ON CEMENT USED IN TESTS TO DETERMINE THE EFFECT OF AIR ENTRAINMENT AS A PROTECTION AGAINST ATTACK BY SODIUM SULFATE

Cement	1	2	3	4
Tensile strength psi				
1–3 Ottawa sand–3 days	295	265	275	270
7 days	360	380	360	380
28 days	440	420	445	480
Compr. strength psi				
1–2 Ottawa sand–7 days	4967	4221	3961	4221
28 days	6396	6104	5877	7013
% Pass. No. 325	93.2	91.9	87.1	86.8
Surface area (Wagner)	1820	1720	1650	1760
% Water	24.5	25.0	23.5	24.0
Chemical analysis				
Oxide analysis				
SiO_2	21.5	21.8	21.5	23.9
Al_2O_3	6.0	5.8	5.6	4.8
Fe_2O_3	2.4	2.7	3.7	3.1
CaO	63.7	64.2	64.5	63.8
MgO	2.1	1.6	1.6	1.3
SO_3	1.7	1.8	1.7	1.7
Ignition loss	1.2	1.3	1.0	0.86
Insoluble residue	0.25	0.19	0.19	0.26
Alkali (Na_2O)	1.08	1.08	1.03	0.70
Compound composition				
C_4AF	7	8	11	9
C_3A	12	11	9	7
$CaSO_4$	2.9	3.1	2.9	2.9
C_3S	50	49	53	39
C_2S	23	25	21	38
Al_2O_3/Fe_2O_3	2.5	2.1	1.5	1.5

The benefit of the air entrainment in the case of the 5.5 sack mix cement No. 2, is not readily apparent from the picture (Fig. A) but again an inspection of all specimens of the same mix shows clearly the superiority of the air entrained concrete.

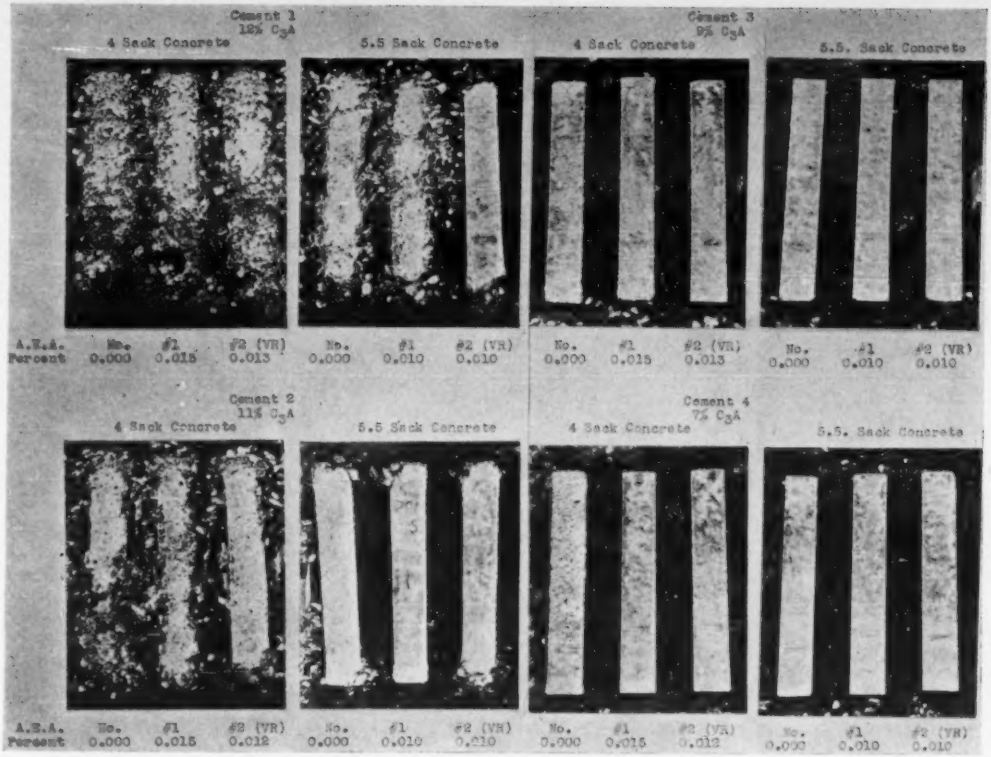

Fig. A—Showing effect of air entrainment in concrete exposed to an alkali soil containing a high percentage of Na_2SO_4. Exposure period approximately one year. Test specimens 6- x 5- x 34-in. beams

TABLE C—TESTS ON CONTROL SPECIMENS (6 x 12 IN. CYLINDERS) OF CONCRETE IN THE AIR ENTRAINMENT SODIUM SULFATE TEST SERIES

[Cemt. No.	percent A. E. A.	w/c	Water lbs. per sack	Yield sks. per cu. yd.	Slump	percent voids ASTM C138-44	Wt. per cu. ft. wet	Set wt. per cu. ft.	Modulus of elasticity 10^6 28 days Sonic	Modulus of elasticity 10^6 28 days Comp. E 1000 psi.	Modulus of elasticity 10^6 1 year Sonic	Modulus of elasticity 10^6 1 year Comp. E 1000 psi.	Compr. str. 28 days psi.	Compr. str. 1 yr. psi.
									4 sack concrete					
1	None	1.145	71.6	3.92	2	2.5	150.4	152.2	4.8	3.8	4.2	3.2	2770	2990
	.015 No. 1	1.115	69.7	3.93	3	5.6	145.5	145.2	3.9	3.0	3.5	2.7	2120	2110
	.013 No. 2 (VR)	1.061	66.2	3.92	2¾	6.3	145.2	145.8	4.2	3.4	3.5	2.6	2360	2420
2	None	1.108	69.3	3.94	2½	2.7	150.6	152.0	5.2	4.0	4.4	3.1	2670	3030
	.015 No. 1	1.063	66.3	3.91	3	6.8	144.4	146.0	4.6	3.4	3.6	2.3	2160	2175
	.012 No. 2 (VR)	1.058	66.1	3.93	3¼	6.4	145.1	145.3	4.5	3.5	3.9	2.8	2285	2305
3	None	1.145	71.6	3.94	2¼	2.1	151.0	151.7	4.7	3.4	4.7	3.9	2375	3220
	.015 No. 1	1.051	65.7	3.93	3½	6.6	144.9	145.5	4.1	2.6	3.8	2.7	1910	2285
	.012 No. 2 (VR)	1.027	64.2	3.95	2⅞	6.4	145.5	147.4	4.8	3.6	4.3	3.1	2350	2830
4	None	1.104	69.0	3.96	2¼	2.2	151.3	152.2	4.0	3.2	4.8	4.0	2565	3340
	.015 No. 1	1.032	64.4	4.00	2¾	5.2	147.2	148.6	4.0	2.8	3.9	2.9	2430	2835
	.012 No. 2 (VR)	1.032	64.4	3.98	2¾	5.6	146.6	148.7	3.9	3.1	4.6	3.5	2450	2935
									5.5 sack concrete					
1	None	0.851	53.1	5.40	2½	1.6	152.4	152.7	5.1	3.7	3.5	2.5	3450	3460
	.01 No. 1	0.808	50.5	5.42	2⅝	5.1	147.2	147.8	4.8	3.9	4.0	2.9	2880	2945
	.01 No. 2 (VR)	0.789	49.3	5.40	2¾	5.6	146.8	148.5	4.9	3.9	4.4	3.3	3575	3305
2	None	0.834	52.1	5.39	2½	2.0	152.0	153.6	5.5	4.9	4.8	4.0	3755	4420
	.01 No. 1	0.789	49.3	5.44	3¼	4.9	147.9	147.8	5.0	3.5	4.1	2.9	2945	3270
	.01 No. 2 (VR)	0.786	49.1	5.39	3	5.9	146.4	146.5	4.9	4.0	4.1	3.2	3335	3430
3	None	0.793	49.5	5.44	2¾	1.8	153.0	153.8	5.6	4.6	5.3	4.3	3985	4650
	.01 No. 1	0.723	45.2	5.48	3¼	5.6	148.1	148.6	5.0	4.1	4.5	3.4	3300	3950
	.01 No. 2 (VR)	0.723	45.2	5.48	3	5.6	148.1	148.6	4.9	4.1	5.1	4.3	3450	3925
4	None	0.796	49.7	5.46	2¾	1.4	153.6	154.4	5.4	4.4	5.9	4.7	4210	5440
	.01 No. 1	0.742	46.3	5.52	2¾	4.5	149.4	150.2	4.6	3.4	5.0	3.9	3570	4695
	.01 No. 2 (VR)	0.742	46.3	5.51	3	4.6	149.3	150.0	4.8	3.8	5.4	4.4	3940	5275

Wet concrete

Set concrete

6 x 12-in. Cylinders

NOTE: Specimens cured 21 days in the moist room and the balance of the time in outside air. Method of cure accounts for the relatively low strengths at one year.

TABLE D—CONCRETE MIX DESIGN—TESTS TO DETERMINE EFFECT OF AIR ENTRAINMENT AS A PROTECTION AGAINST ATTACK BY SODIUM SULFATE

Materials in 4.5 cu. ft. batch of concrete

Materials	Spec. grav.	4 sks. per cu. yd. concrete		5.5 sks. per cu. yd. concrete	
		Regular concrete	Air entr. concrete	Regular concrete	Air-entr. concrete
Cement	3.15	62.7 lb.	62.7 lb.	86.2 lb.	86.2 lb.
Fine agg.	2.70	257.5	235.0	230.9	208.4
⅜ in. to No. 4	2.77	88.2	88.2	85.0	85.0
1½ in. to ¾ in.	2.77	234.7	234.7	248.4	248.4
Water	1.00	46.0–47.7	42.8–44.2	45.4–47.7	41.4–45.2

Sieve analysis of aggregates

Sieve size	percent passing		
	Fine aggregate	⅜ in. to No. 4	1½ in. to ¾ in.
1½ in.			98
1 in.		100	53
¾ in.		97	12
⅜ in.	100	21	1
No. 4	90	1	
8	81		
16	68		
30	49		
50	19		
100	5		
200	2		

Aggregate weights based on saturated surface dry condition.

By M. SPINDEL*

From the previous papers and the 14 contributions to the Symposium it appears that both the manufacturers and the users of portland cement and concrete are in full agreement regarding the advantages to be obtained by entraining a limited percentage of air into concrete. By limiting the air content to 3 to 6 percent many well-known experts have shown that the matter of air is now well under control and that no serious disadvantage will occur from entrained air if the above limit is not exceeded.

The writer agrees in principle but wishes to add that since "entrained air alters many of the basic properties of concrete mixes" and of the hardened concrete as well, a complete analysis and balance of the advantages and disadvantages resulting from the entrained air is still necessary or at least desirable. The valuable data that have been established for many years in the laboratory and in the field establish, without doubt, a sound basis for still further investigations. The writer,

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who had developed and published for many years some special methods for the design of normal high quality concrete, has not yet been in a position to make a similar contribution regarding air-entrained concrete but perhaps he might again be permitted some remarks to the problem in general and to some of the contributions to the Symposium especially.

Regarding the analysis of advantages and disadvantages resulting from entrained air, we have to bear in mind that the resistance to scaling of the surface of concrete roads treated in winter with calcium chloride to remove ice, is not the sole measure of durability of concrete and not even an uncontested measure of resistance to frost in normal circumstances, i.e. if used for structures which are not treated with corrosive chemicals in the same way. From tests carried out recently in Sweden (Highway Research Abstracts, JOURNAL ACI V. 17, No. 6 June 1936) it may be seen that calcium chloride, which proved to be beneficial to concrete when used as an admixture, is highly corrosive to hardened concrete when used for removing ice.

When designing concrete for various types of structures we have, therefore, always to ask ourselves whether there is any reason to sacrifice a more or less high percentage of the various strengths including bond to reinforcement which might be badly needed, in order to get instead a high resistance to scaling by the action of calcium chloride and frost which will not occur to the structures in question. But even for roads the various strengths must not be sacrificed if not absolutely necessary, and above all the resistance to abrasion is, without doubt, one of the most important properties of roads of every description which must not be overlooked.

In the contribution by H. L. Kennedy the author has shown how very much entrained air reduced the resistance to abrasion, especially on the top of the specimens which may be compared best with the surface of concrete roads. Like Mr. Kennedy, I too, have to repeat my warning in various discussions that entrained air, if not controlled sufficiently, will give concretes very low in strengths, resistance to abrasion, etc.

Some experts on concrete might have been surprised by the results of a series of tests "that there is little difference between the homogeneity of an air-entrained concrete and a relatively high-bleeding concrete, despite the fact that in the case of air-entrained concrete the bleeding is reduced greatly or eliminated altogether." If we consider the advantages and disadvantages more fully we shall have to ask ourselves whether there are no other means and types of admixtures available which combine sufficient resistance to frost with even increased strengths, bond to reinforcement, resistance to abrasion, etc. It is well-known that such properties can be obtained by a reduced water-cement ratio and the lowest (water plus air) cement ratio possible, and many col-

leagues including the writer would prefer the future development rather in this direction.

The above general remarks do not mean that the valuable achievements with air-entrained concrete were not appreciated sufficiently by the writer, but what is still needed is a clearer common basis regarding the design and the properties both of normal and air-entrained concrete as may be seen from the following special remarks.

In the contribution by W. A. Cordon, Fig. 1, 2, 2a and 3 show which reduction of "water content—lb. per cu. yd." can be obtained by "percent entrained air (by abs. vol.). By calculating from the water content the percentage of water by absolute volume one learns that the reduction of 6 lb. or 8 lb. or 9 lb. water content per cu. yd. for one percent entrained air means a constant water reduction of about 0.36 percent or 0.47 percent or 0.53 percent of water for every 1 percent of entrained air, both per unit of fresh concrete. That means, of course, in all circumstances an increased percentage of the absolute volume (water plus air) and, therefore, also an increased (water plus air)/cement ratio which results and has to result in a decreased compressive strength. Without further details one cannot understand why for the lean mix of 1.13 bbl per cu. yd. cement content an increased compressive strength was obtained for every percent of air up to 3 percent and no decrease in compressive strength even up to 6 percent of entrained air, as has been shown by the author in Fig. 6a. Although these very remarkable results are in accordance with similar results shown by other authors, for example in Fig. 1 of the contribution by Walker and Bloem which will still be discussed, some more details and explanations would help to avoid misunderstandings. At any rate it might be of great advantage if all concrete constituents were given also by absolute volume per unit of fresh concrete.

The same is true regarding the example of computation of trial mixes based on the excellent ACI Standard Recommended Practice, where the absolute volumes of the various constituents per unit fresh concrete are given, so that only the percentages have to be calculated. Although the author showed in this example quite correctly how the mix for air-entrained concrete and constant cement content could be redesigned simply by replacing 3 percent of sand by abs. vol. by 3 percent of air the writer is of the opinion that this simple way of adjustment might cause some misunderstandings or even mistakes. The air volume should be considered in principle as a quite separate matter so that the concrete mix and the properties of the concrete could be controlled by the fundamental law given by the voids-cement ratio (V/C) and the cement-space ratio $\left(\frac{C}{V+C}\right)$ which were used in the formula by Talbot and

Richart over 20 years ago and have been proved to be valid also for air-entrained concrete by H. F. Gonnermann and others (JOURNAL ACI V. 15, No. 6, June 1944).

It is, without doubt, an advantage that Mr. Cordon's results were not based on the slump test alone but also on workability tests as to Fig. 5, but Mr. Powers' remolding test was not intended for many hundreds of jigs if carried out in the usual way. It might be of interest to learn whether the results of the remolding test with crushed aggregate were considered reliable by the author.

The contribution by Walker and Bloem brings some more very interesting results but here, too, it might have been an advantage to show both the entrained air and the water content, and cement and aggregate as well, in their percentages per unit of fresh concrete by absolute volume. Very interesting are the results in Fig. 4 showing that the reduction of mixing water per unit is not constant for every percent more of entrained air as has been shown in Fig. 1, 2, 2a, 2b and 3 in the contribution by W. A. Cordon. It is of great importance to know that "as more air was entrained it became progressively less effective in reducing mixing water and also the air was progressively less efficient as the cement content was increased."

The above results confirm again that only a very few percent of entrained air will reduce the water content to a degree that the percentage of water plus air and the (water plus air)-cement ratio is not highly increased. Further it confirms the previous results obtained by Kennedy and the writer that entrained air can not reduce the water cement ratio of neat cement paste nor can it improve the plasticity of neat cement paste for the same water-cement ratio, and that entrained air reduces, without doubt, the water-cement ratio of neat sand-water mixtures, much more than could be achieved by a different fineness modulus of sand or total aggregate or in any other way.

By W. A. CORDON

Mr. Spindel's comparison of Fig. 1, 2, 2a, 2b, and 3 of my paper with Fig. 4 of the paper by Walker and Bloem is not applicable. Mr. Spindel neglects to consider that the curves in my paper are plotted from mixes containing a constant w/c ratio, whereas the curves by Walker and Bloem are based on mixes with a constant cement content and show that mixes with different cement contents have different water reduction values. Mixes from my paper containing a constant w/c ratio will produce a horizontal line on Walker and Bloem's Fig. 4 as the cement content decreases with an increase in entrained air indicating that the percentage reduction in water is constant.

By HENRY L. KENNEDY*

Mr. Spindel's discussion indicates the possibility that some confusion of thought has arisen regarding the similarity of air entraining concretes and high bleeding normal concretes in regard to homogeneity. The only characteristic in common between these two types of concrete is their apparent lack of homogeneity, all other properties of the concretes being markedly different. The lack of homogeneity in these two concretes is due to entirely different factors: 1) in the case of high bleeding normal concrete being due to migration of the mixing water toward the top of the specimens, and 2) in the case of air entraining concrete probably but not definitely known as being due to migration of the entrained air toward the top, possibly aggravated by gravitational settlement of the heavier constituents of the concrete.

The author wishes to emphasize that the above observations appear to be significant only in cases of 10 per cent entrained air or more. Also the lack of homogeneity and resistance to abrasion of concrete containing less than 6 per cent air was found to be little or no different than for plain concretes when the cement contents of the comparative concretes were maintained constant. It should be borne in mind that no density measurements were made and that conclusions regarding homogeneity are based entirely on abrasion tests.

In conclusion, the author wishes to reiterate his belief that improved resistance to $CaCl_2$ or other salts and improved resistance to alternate cycles of freezing and thawing are not the only important benefits to be derived from the use of air entraining agents in concrete. Improved workability which makes for far greater ease of placement of concrete is equally important and can be utilized not only to produce better concrete in place but also to decrease the cost of concrete placement in the field, provided, of course, that air content is closely controlled and does not exceed 6 per cent by volume. Though other additions may be discovered that will do a similar job without any decrease in density, the fact remains that we have a valuable tool today in air entrainment which when judiciously used is beyond reasonable doubt the outstanding contribution to concrete technology in many years.

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The Repair of Concrete: An Introduction*

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Member American Concrete Institute

SYNOPSIS

The repair of concrete structures is an engineering problem, each job containing the elements of diagnosis, treatment and execution. Diagnosis is essential to devising successful repair. Treatment may mean the correction of faults of design, materials, workmanship; protection against destructive agents and exposure; restoration of decay; or a combination of these. The execution of repair may sometimes use methods of expediency rather than logic—a compromise between what one would like and what one can do. The paper considers the more common agents destructive to concrete and is a brief introduction to an important subject.

The repair of concrete structures is an engineering problem and should be dealt with as such. In some cases the problem may be simple, in others complex, but whichever it may be, it can be resolved into a few basic elements each of which needs to be considered in any repair. This paper is an attempt to outline briefly some of these. In general, all repair jobs, whatever their nature or complexity, contain these elements:

(a) Diagnosis—the determination of the cause for the deterioration.

(b) Treatment—the choice of a method which will stop further deterioration and restore the structure.

(c) Execution—the application of the treatment adopted.

Diagnosis

It is not uncommon to find repair jobs being undertaken with little or no thought given as to why the concrete in question came to need repair. It hardly seems necessary to point out that any repair stands a better chance of being successful if it is predicated on an understanding of what brought about the deterioration, for without this it often happens that while the remedial measures adopted may restore the concrete for

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a time, deterioration is likely to reoccur for the same reasons that caused the original trouble. Therefore, diagnosis is an important element in devising a successful repair.

Diagnosis presupposes the ability to determine from various phenomena, by inspection or experiment, the underlying causes for the conditions observed. Applied to concrete, this requires a knowledge of the agents that attack it and the manner in which they act. The destructive elements may be from natural causes such as weathering, from conditions of service such as abrasion, from stresses developed within the structure such as settlement or from a combination of all three. To determine which may have damaged a given concrete requires a study of the form and occurrence of the damage and the manner in which it came about.

Because the deterioration of concrete occurs in many forms and for a variety of reasons and because the same outward form may result at different times from different reasons, it is difficult to develop a wholly satisfactory classification under which the causes of deterioration can be systematically studied. Therefore, no attempt will be made here to present a classification and the groupings that follow are offered simply to facilitate discussion.

One approach to the problem would be to consider all deterioration as due to some deficiency in either design, materials or workmanship. This grouping does not take into account the deterioration that sometimes occurs solely by reason of exposure or misuse nor does it differentiate cause from effect. Both the reason of the failure and the mechanism that brought it about need to be considered before deciding on the remedial measures to be taken in any given case.

Faults of design

Consider for a moment the defects that may originate with design. Most of these cause cracking in one way or another and a study of each particular case will usually reveal which of several possible causes is to blame. And it should be kept in mind that with outdoor concrete, the structural origin of the defect may be obscured by the effects of weathering. A few of the more common sources of structural cracking are:

(a) Foundation troubles such as settlement or other movements.

(b) Insufficient reinforcement at openings as over doors, around windows, at the corners of pits, stair wells, gate openings in dams, etc.

(c) Concentration of embedded pipes and conduits in floor slabs and walls.

(d) Poor details where embedded parts such as rails, steel beams, pipes or gate checks are incorporated in the structure.

(e) The omission, improper placing or poor design of contraction and other joints.

(f) Extreme changes in section.
(g) Overloading.

Under troubles due to design may also be classed the many that result from a lack of proper drainage, either within or around the structure. It is usually accompanied by one or more of the following—efflorescence, calcium carbonate deposits or ravelling—and lack of drainage should be suspected where any of these phenomena are observed. It is well to check a structure to see if there are any pockets where water is being trapped, if drains called for in the drawings were omitted or if they have become inoperative due to plugging or other cause.

Spalling may be due either to design, workmanship or materials but the designer has the first responsibility, except where materials alone are at fault. Some of the more common causes of spalling are:

(a) *Pinching of joints.* The exposed face of a slab such as a deck or wall will often expand more than does the slab as a whole, and as a result pinching and spalling may occur along the edge of a joint.

(b) *Drag at joints.* Slabs intended to slide on the supporting abutment may fail to do so, and if the binding occurs near the outer edge of the bearing surface it may spall the edge of the abutment below. This can also occur if proper care has not been taken in the construction of the joint.

(c) *Rusting reinforcement.* There may be spalling due to rusting of the embedded steel if it has not been provided with a sufficient cover of concrete for the exposure encountered. Similar spalling can be due, also, to a poor grade of concrete, the result of careless workmanship.

(d) *Popouts.* When unsound aggregate particles lie close to a concrete surface they will, with certain exposures, cause unsightly "popouts" or spalls. These will be bigger and deeper the poorer the concrete or the larger the size of the aggregate involved.

Faults of materials

The quality of concrete is dependent on the quality of its constituents which are cement, aggregate and water. Defective cement or aggregate usually cause volume changes that are evidenced by pattern cracking, more or less conspicuous. Although not strictly true, in general the quality of the mixing water seems to have little influence on the durability of the hardened concrete in which it is used but the quantity of mixing water is most important: the latter is a matter of workmanship and will be discussed under that heading.

Defective cement is a relatively rare cause of deterioration and when it occurs, usually results in complete failure or is accompanied by symptoms so unusual as to suggest the need of a thorough investigation. Some authorities blame nearly all defective concrete on deficiencies in

the cement used; others claim that only a few failures can be traced to this cause. Undoubtedly the truth lies somewhere between. Cement is probably a secondary or contributing factor in many cases for which other causes are assigned. But it acts only as other factors, such as poor workmanship, make a concrete liable to attack by the normal agencies of destruction. Insofar as the diagnosis of concrete ills are concerned, unless gross volume changes are encountered cement as a cause of deterioration can be largely ignored and its effects considered under other headings.

Stanton has pointed out a type of failure in which cement does play an important role. Cements high in sodium and potassium, when combined with certain types of silicious aggregates, have been found to cause concrete to expand, resulting in conspicuous cracking. This expansion is accompanied by other well-established phenomena such as the formation of certain reaction products of which sodium silicate is one. The reaction has been given much publicity in the technical press and the reader is referred to the many excellent articles in the JOURNAL and elsewhere for details of its occurrence and methods for its identification. In localities where the cements are high in the alkali components and where potentially dangerous types of aggregate might occur, any unusual cracking or other evidence of volume change should be investigated with this in mind.

Expansion similar in appearance to that just discussed, may be caused by the aggregate alone. It is usually due either to unsoundness of some of the aggregate particles or to major differences in the coefficient of expansion of the aggregate and the surrounding concrete. Some spalling may accompany unsound aggregate and give a clue to the cause but differences between the coefficient of expansion of the aggregate and concrete are harder to detect since the aggregate itself is sound.

Poor grading might also be considered a fault of the aggregate and strictly speaking it is, but it affects the concrete not through any lack of quality of the particles themselves but because of an increase in the porosity of the concrete in which it is used. The defects that arise from this cause are similar to those discussed under workmanship and will be considered there.

Faults of workmanship

Defects caused by workmanship are many and varied; they are also by far the most numerous and account for or contribute to probably ninety percent of existing deterioration. Any omission, inaccuracy or carelessness in any of the steps that go into the preparation of the materials, the proportioning, mixing, placing or finishing of concrete or its protection afterward has a deleterious effect on its quality and makes it less able to resist weathering or other destructive agent. It is without

the scope of this paper to discuss in detail the many faults of workmanship; they have been described many times in the pages of the JOURNAL, but for the sake of completeness they will be briefly listed.

(a) *Poorly graded aggregates.* Either inherent in the aggregate itself or brought about through segregation in handling or in stockpiles and bins.

(b) *Improperly proportioned mixes.* Either due to a poor mix design or inaccurate measurement of the materials.

(c) *Excess water.* This could be classed under (b) but because it is a major cause of poor concrete it has been listed separately. No single defect of workmanship has caused so much inferior concrete as the use of an unnecessary amount of water in mixing.

(d) *Insufficient mixing.*

(e) *Segregation or separation of the constituents of the concrete.* It takes place during handling and placing of the concrete and may be due to carelessness in these operations. It may be made almost a certainty, even if carelessness in placing does not occur through any of the faults of workmanship listed from (a) to (d).

(f) *Over-manipulation.* Either through excessive puddling, vibration or trowelling or from finishing at the wrong time.

(g) *Lack of curing and protection.* Such as allowing the concrete to dry out too soon, permitting it to freeze, etc.

In general, it may be said that faults of workmanship do one of two things: either by increasing the porosity of the concrete make it more susceptible to deterioration, or by interfering with the hydration of the cement weaken it and make the concrete less able to resist attack. Thus, with this class of faults it is more important to know the forces acting to destroy the concrete and the way they act than to determine which of several faults may have given rise to the particular defect to be corrected.

Destructive agents

An entire paper could be devoted to a description of these agents and how they act, but for our present purposes it will be sufficient to indicate the more important. For simplicity they will be divided roughly into three groups: those that act on the surface of the concrete, those due to moisture movements within the concrete and those that cause volume changes.

Under the first group, those that attack concrete at its surface are erosion, abrasion and corrosion. Each of these brings about a wearing away of the surface; the first by wind and water, the second by abrasive or mechanical means and the third by chemical action. To these could be added scaling caused by frost acting on a surface that has been overworked.

Moisture movements require that concrete contain pores or channels through which water or vapor can pass. Movements may be brought about by absorption or evaporation, they may be due to pressure forcing water from a wet to a dry surface as in a wall with water on one side, or moisture may be drawn from a wet to a dry face by the forces of evaporation and capillarity. The moving water dissolves some of the constituents of the hydrated cement, thus weakening the concrete, and it may deposit the products of this solvent action beneath the surface so as to cause scaling. If also the moving water contains corrosive chemicals, the rate of attack is greatly accelerated and if the pores become saturated at a time when a falling temperature causes the water in them to freeze, the concrete may be destroyed through frost action.

The forces causing volume changes are many. Temperature is one, loss of moisture is another, defective materials a third, and another, chemical changes such as that brought about through the action of the sulfates of sodium and potassium on concrete or the alkali-aggregate reaction already mentioned.

Exposure

One other factor needs to be considered before passing on to the question of repairs, namely that of exposure. Structures, in contact with moisture on one side only, such as dams or retaining walls, are subject to a much more severe exposure than those normally dry, such as the superstructure of a building. The south and west faces of dams and other water-impounding structures are, on this continent, more liable to deterioration than those facing north or east; and in the same structure, horizontal surfaces more than vertical surfaces. The severity of an exposure can be roughly gauged by the opportunity offered to the concrete to become saturated with moisture and the number of annual cycles of freezing and thawing to which it is subjected. Time is an important element of exposure. For instance, surface scaling in a structure only a couple of years old would indicate a concrete of poor quality, a very severe exposure or both; the same degree of scaling in a concrete twenty-five years old would not be very significant.

Methods of repair

Let us consider in a general way, the problem of repair. It seems to the author that any repair should

(a) Correct structural defects, if any, and strengthen the structure, if weakened.

(b) Eliminate, if possible, the sources of deterioration.

(c) Restore the decayed parts of the structure to its original form.

(d) Protect the concrete, if necessary, to prevent further damage.

It hardly seems necessary to argue the desirability of correcting structural defects, if serious, prior to reconditioning the concrete. The problems likely to be encountered are many but they are structural rather than a matter of the quality of the concrete, and will not be discussed here. In general, it may be said that they require the highest engineering skill for their solution.

Elimination of the sources of deterioration is not always possible. It is obvious that a structure cannot avoid the normal hazards of service such as natural weathering. But in many cases it is possible to remove the sources of deterioration. A typical example of this would be faulty drainage, one of the more common faults encountered. It is often possible to intercept water reaching a structure by rearranging the drainage in the surrounding area so that the concrete will not come in contact with it. Water trapped in the structure can be piped away and not allowed to seep through cracks, to run down surfaces or to remain on floors, platforms, or decks. But if the cause of deterioration cannot be eliminated, then the structure must be otherwise protected.

Restoring the decayed parts of a structure may be either simple or difficult. If all that is involved is the removal of the defective concrete and its replacement, the problem is simple and its execution only requires the application of well-established methods of construction. But if it requires the removal of load-bearing sections or of keeping structures such as a dam or bridge in operation, it may become a major engineering problem. In the latter case, the methods of repair have often to be those of expediency rather than logic—a compromise between what one would like and what one can do.

Concrete can be protected from damage in various ways. One is to eliminate the cause of deterioration but, as has been pointed out, this is not always possible. In such cases, re-surfacing of the affected concrete offers a solution, using pressure-applied mortar, plastering or concrete cast in place. Usually the thicker surfacings are the more permanent. Such surfacings should be reinforced and dowelled to the structure they cover. Care must be taken to prevent water from being trapped behind the surfacing else the results may be disappointing. Re-surfacing can be carried a step further until the original structure is totally encased; where this protects the original concrete against further attack, it is a satisfactory method of handling many jobs, especially with massive structures. Leaky dams and other water-impounding structures require that some means of cutting off the seepage be provided. A cut-off wall or blanket of concrete on the upstream side is a very satisfactory solution to such a problem, although not always the cheapest. Where the structure is part of a storage system or hydro-electric development, this

is not always possible in which case grouting of all joints and fill planes will usually dry up the structure to a reasonable degree.

In closing, the author would repeat that this paper is not intended to be either complete or exhaustive but only an introduction to an important subject which to an increasing extent is engaging the attention of engineers throughout the continent. Its purpose will be served if it helps engineers to a better understanding of the problems of deterioration and repair and causes them to study the many admirable examples of successful work that have been reported in the literature wherein the principles outlined have been applied.

[To facilitate selective distribution, **separate prints** of this title (42-40) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946.]

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Behavior of Concrete Structures Under Atomic Bombing*

By E. H. PRAEGER†
Member American Concrete Institute

SYNOPSIS

The destruction wrought by atomic bombing of the Japanese cities, Hiroshima and Nagasaki, August 1945 is outlined, with an analysis of typical damage within areas with respect to "zero point." The survival of certain modern buildings of reinforced concrete and composite construction is noted with interest. The paper discusses principles and procedures of design necessary to resist attacks by these special new weapons.

INTRODUCTION

Shortly after the end of the war in Japan the Chief of the Bureau of Yards and Docks recommended to the Secretary of the Navy that a group of qualified officers and technical experts be assigned to special duty to survey damage wrought by the atomic bombs and other weapons on targets in Japan. It was anticipated that criteria would be developed from the observed facts for the design of military structures that would withstand present and future special weapons.

Surveys were made of targets that had been attacked by atomic bombs, high explosive bombs of various sizes and incendiary bombs. Targets of bombardment by shells of naval guns were not visited because another Navy group had been assigned to these duties. However, information concerning the physical damage inflicted by this weapon was received from that group.

In addition to the data from inspections of damaged structures, considerable information of interest was obtained through interviews with Japanese who were in the target areas at the time of the bursts and with city officials, architects, engineers, contractors and owners of

*Presented 42nd ACI Convention, Feb. 20, 1946.
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damaged buildings. Original plans of many of these buildings were obtained from the designers and several objects and samples of representative materials were shipped to the United States for tests.

ATOMIC BOMBS

Two atomic bombs were released over Japan, one over the city of Hiroshima and the other over Nagasaki. Both bombs were detonated in the air.

Hiroshima

The Hiroshima bomb was released at 8:15 a.m. on August 6, 1945 over the center of the city.

Hiroshima was a commercial center situated in a delta at the mouth of the Ota River. It comprises five islands extending in a north-south direction and the adjacent mainlands. The city proper is approximately four miles wide and five miles long, the various islands and mainland being interconnected by many bridges. The topography is level within the city limits but hills rise to an elevation of about 800 feet to the northwest and northeast.

While there is a shipyard and several industrial and military facilities on the outskirts of the city, the center contained many parks, shrines, residences, schools, military reservations and a considerable number of multi-story commercial buildings. The population in July 1945 was approximately 350,000.

It has been stated that the explosive peak pressure of this bomb was equivalent to that of 10,000 tons of TNT. Observation of the damage indicates, however, that this figure may be high. The location of the burst was determined by projecting edges of flash burns, defined by shadows of intervening objects, and found on numerous timber and other objects, to a point of intersection. The point on the ground directly below the burst has been designated as the "zero point".

The explosion has been described as a red flaming meteor-like ball of fire; thereafter a great sudden pressure and heat; then noise. A cloud of dust filled the area, and rain fell in torrents in some sections of the city, accompanied by a wind.

Fires of secondary origin sprang up in all directions and burned for a considerable period as no attempt was made to extinguish them. The momentary heat at the ground surface has been estimated, by the Japanese, to be in the range of 2000 C. Tests of sample roof tiles exposed to the blast heat are now in progress at the National Bureau of Standards.

Approximately one quarter of the population were killed immediately or died within the next several weeks, one quarter suffered injuries but

survived and, of the remaining half, many lost their homes and became ill because of exposure and other hardships.

A medical team reported that no radioactivity was found in the ground two weeks after the explosion.

In general appearance the center of the city resembles the completely burned out areas of Tokyo and other Japanese cities which were targets of large-scale incendiary raids. Destruction of small, lightly constructed dwellings and business structures was complete, and fires consumed practically all such buildings within an area having a diameter of two miles with the zero point as a center. Light steel and timber structures collapsed or suffered various degrees of failure in an area of approximately ten square miles.

Brick, concrete block and stone masonry structures were completely destroyed within an area extending some 5,000 feet from the zero point.

The majority of the modern earthquake proof buildings of reinforced concrete, fireproofed structural steel, or composite construction (of structural steel and reinforced concrete) at distances greater than 2000 feet from zero point suffered little or no damage to the main structural parts. Closer to zero point a number of these buildings suffered some structural damage. Few such buildings collapsed but many were gutted by fire.

Bridges suffered varying degrees of damage, some by forces lifting the deck. Piers and other water front structures were outside the sphere of influence of the bomb.

Reinforced concrete chimneys suffered relatively little damage. Underground utilities generally were not damaged but above ground wiring systems were put out of service because of failures of supports. Steel tanks containing gas and liquid fuels were damaged beyond use at distances of 6000 feet from zero point.

Few trees were blown over but those near the zero point had been burned and in many instances heavy branches were broken.

The effects usually connected with high wind pressures were not evident except at the fringes of the devastated areas. While doors and windows on surfaces exposed to the blasts were generally blown in, many cases of inward, rather than outward pressure were noted also on the leeward side of structures. This condition has also been observed with high explosives; the velocity of the blast wave is so great that the pressures occur almost simultaneously on all sides of an obstruction. Evidence of downward pressure on roofs near the zero point was common.

The survival of the many modern buildings of reinforced concrete and composite construction of from 3 to 10 stories, is of extreme interest. While roofs of buildings close to zero point were deflected downward

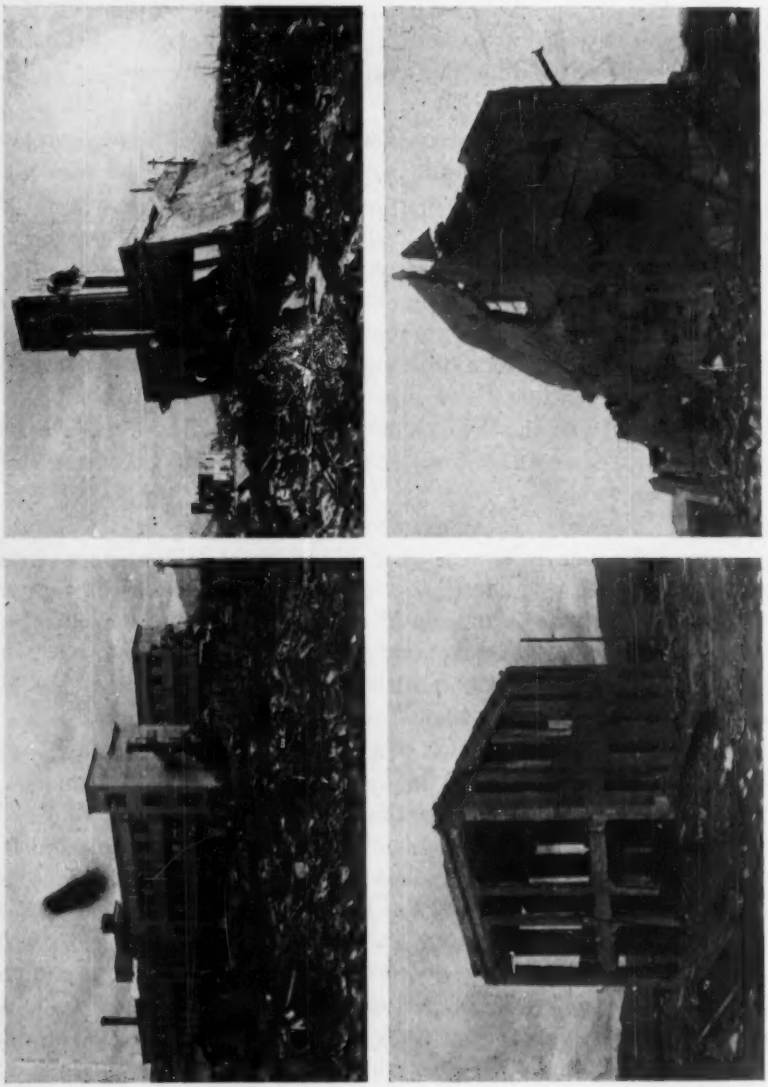

Fig. 1 — Hiroshima after the atomic bomb: a (upper left), Red Cross Hospital; b (upper right), Shimomura Watch Co.; c (lower left), south view of Bussan Shohin Buretsu Kan; d (lower right), Hiroshima Ice Manufacturing Co.

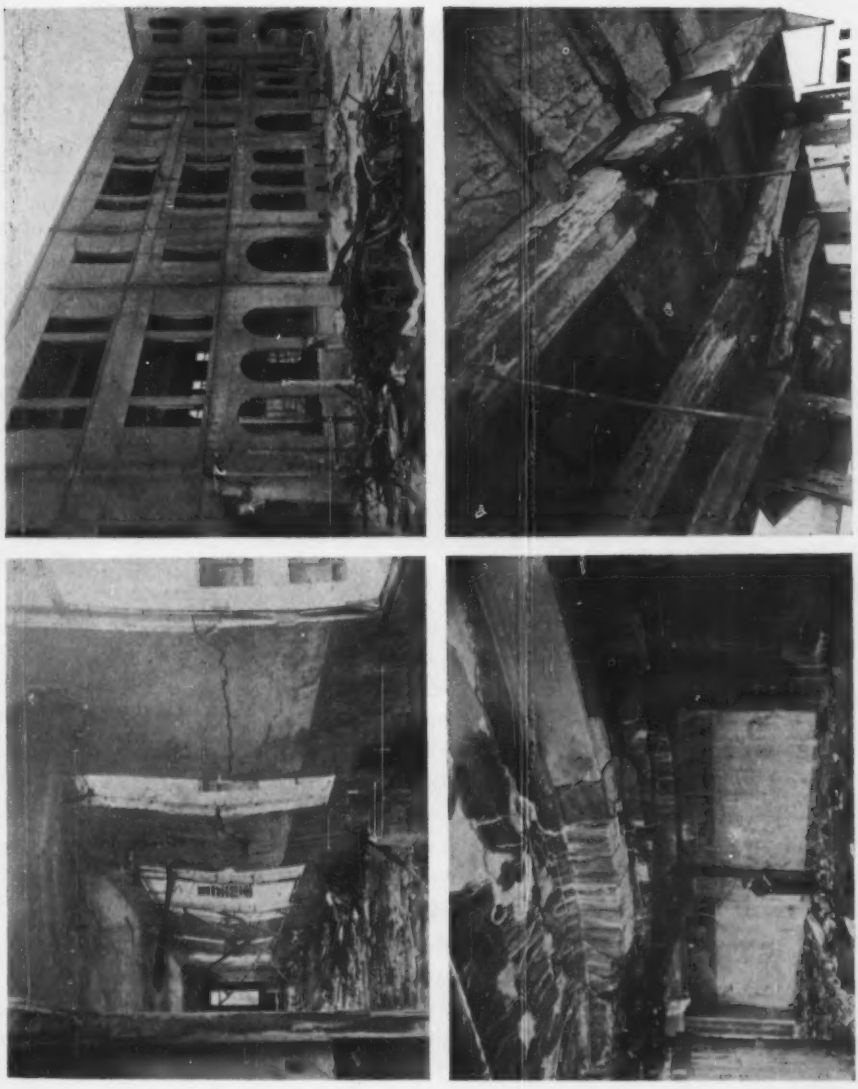

Fig. 2—Hiroshima structures after the blast: a (upper left), Honkawa National School, b (upper right), Honkawa National School, c (lower left), Fuel Distribution Building, d (lower right), roof of Bussan Shohin Bureto Kan.

and local failures were experienced in many members, the majority of these buildings can be restored to their original condition. In these buildings fires, probably most often started by burning embers from adjacent buildings, caused considerable damage. It has been reported that most fires in fireproof buildings did not start until about twenty minutes after the burst.

Fig. 3—Hiroshima Teikoku Bank, after the blast.

Nagasaki

Nagasaki is an industrial city with a reported population of approximately 250,000 in July 1945. The city lies on both sides of Urakami River and Nagasaki Bay into which the Urakami flows. The center of the city is approximately at the mouth of the river and the city extends from this point about two miles in a northerly and two miles in a southerly direction. Hills, approximately 1200 feet at their highest peaks, parallel the river on both sides. The valley varies in width from one half mile at the north to two miles at the south. Industrial plants, business sections, residential areas, schools, hospitals and other facilities were located in the valley on both sides of the river.

The bomb burst at 11:00 a.m. on August 9, 1945. This bomb was of a type different from that released over Hiroshima and was equivalent in characteristics to a theoretical bomb with a 20,000 ton charge of TNT. The appearance of the burst, insofar as could be learned from discussions with Japanese, was similar to that for the bomb released over Hiroshima.

The general appearance of the section of the city, for a distance of one mile north and one mile south of zero point, is ghost-like. Burned ruins predominate. Yet many permanent type buildings survived with comparatively little structural damage. Destruction extended in a northerly and southerly direction for a distance of approximately two miles from the zero point but was confined in an east-west direction to the area between the hills paralleling the river and bay.

In an area within 7,000 feet of zero point all small frame buildings, dwellings, stores, etc. were completely destroyed leaving only foundations and broken roof tile. The remains of these buildings were destroyed by fire. Trees were tilted in the direction away from the bomb and many were uprooted. Doors and windows, including frames and glass were blown inward on the side facing the bomb, but as a rule outward on the opposite side. Contents of buildings, except heavy machine tools, showed evidence of having been blown about. Burning of inflammable materials in fireproof buildings was widespread but not complete. Wood floors, interior trim, furniture, etc. were found to have been entirely consumed in one room and unburned in the next which leads to the conclusion, confirmed by eye witnesses, that many fires, particularly those in fire-proof buildings, were secondary rather than primary.

As would be expected, heavily built steel structures suffered damage to a lesser degree than the comparatively light factory type steel buildings. Light steel and timber structures, were severely damaged by blast pressure and primary or secondary fires. Many buildings of this type were deflected bodily and lean with an inclination of as much as 30 deg.

There was no crater nor evidence of any great disturbance in the ground under the point of detonation.

Deaths and injuries to personnel in the exposed area were in about the same proportions as those noted for Hiroshima. However, a smaller percentage of the total population of the city was in the exposed area.

EFFECTIVENESS OF ATOMIC BOMB

The atomic bombs damaged structures by reason of:

(a) Great pressure, causing collapse or structural damage.
(b) Extreme heat, causing fires.

The atomic bombs produced casualties to personnel by any of the following causes or by any combination of them:

(a) Injuries from collapsing structures.
(b) Flying debris.
(c) Fires.
(d) Great pressure.

Fig. 4 — Atomic bomb aftermath at Nagasaki: a, (upper left), Bridge about 2500 feet north of zero point; b (upper right), Building at Nagasaki Medical School; c (lower left), Building at Mitsubishi Steel and Arms Works; d (lower right), Mitsubishi Shipbuilding Co. office building.

(e) Burns by direct heat rays.
(f) Internal injuries by penetrating "gamma" rays.

The atomic bomb disrupted public utilities over a considerable area. Overhead telephone, light, power and trolley wires were damaged beyond use. Automobiles, trucks, trolley cars and railroad equipment were damaged in varying degrees. Machine tools were dislocated from their bases and damaged by falling debris and subsequent exposure, but generally not beyond repair.

ANALYSIS OF TYPICAL DAMAGE FROM ATOMIC BOMBS

As many of the well built structures in both Hiroshima and Nagasaki, particularly those of composite construction, withstood the blast pressure with comparatively little damage, the question naturally arises: why did these buildings not fail? After a study of many of those structures, one finds some plausible answers to this question.

The spectacular case of the reinforced concrete chimneys, the majority of which survived at all distances from the blast, will be considered first. These structures are of simple standardized design, which probably included allowance for earthquake forces. They are usually built by specialists and are therefore of relatively good construction. Being circular in plan, they offer minimum resistance to the blast wave. Their periods of vibration are relatively long. A common size chimney has an average diameter of 5 ft. The blast wave travels at a rate in excess of 1000 feet per second and therefore the wave traveled from the directly exposed surface to the back in about 1/250 of a second, at which time the entire structure is subject to approximately equal pressures in opposite directions except that there is a secondary effect in the form of a gust of high wind. While the period of the blast is relatively long, it is probable that the impulse passes before the chimney which has been set in motion or has had time to reach the amplitude of its swing. These conditions explain, in general terms, why the concrete stacks withstood a much greater transient load than would be possible were the force an equal static load.

It has been noted that well built reinforced concrete buildings, and buildings of composite construction (there were no multi-story steel frame buildings) not far from the zero point, survived except for damage to some structural parts. In most cases the buildings are well designed, earthquake forces have been included and the materials and construction are adequate. An average size for this type of building might be 100 ft. square and 40 ft. high. The period of vibration of the structure as a whole is longer than are those of most of the component units (slabs, walls, etc.). The structure has a much greater relative depth than that

of the chimney, yet it requires approximately 1/12 of a second for the blast wave to travel from the windward to the leeward face. If the exterior surfaces had no openings and if they did not fail, the external pressures on the structure as a whole would be approximately balanced in 1/12 of a second. The resultant pressure on an outside wall or roof would be the difference between the interior pressure and the blast pressure. If this difference is great enough, the units of insufficient strength would fail but the building as a whole would not necessarily suffer. Under this condition the floor slabs, beams and girders act as compression members, stressed by the external horizontal load and must be of sufficient strength to withstand these forces. Since these parts of the structures are designed for the generally more severe condition of bending, it is entirely possible that they would be strong enough to resist this horizontal thrust.

As a matter of fact, there were relatively few totally enclosed structures, and most of the buildings have a large percentage of window surface. The windows and frames, being relatively weak, are blown in instantly on the windward side. They may be blown out or in on the leeward side depending on whether the blast from the front (through the openings in that wall) or from the back reaches the rear surface first, and both conditions were noted in Japan. The loads on the roof members and on the floors above grade are then equalized by approximately equal pressures above and below except in the case of the floor directly over an enclosed basement, which is subject to unbalanced blast pressure from above. Cases of failure from downward pressure were noted at first floor levels. Cases of both downward and upward failures were noted in roofs although most roof failures, particularly in buildings directly under the blast, were downward. Few failures were noted in floors between the roof and first floor, but this would be possible if the wall openings in one floor were materially different from those in adjacent stories.

PRINCIPLES OF DESIGN

The design of structures to withstand atomic bombing presents many problems. Permanent buildings would necessarily be of fireproof construction and, therefore, the hazard of fire damage to the structure proper would be eliminated. Extremely vital structures that must be constructed in the open should be designed to withstand direct hits of high explosive bombs and shells and the blast pressure of atomic bombs.

The strength and thickness of structures which will be required to resist heavy HE weapons would probably be sufficient to withstand the blast pressures from atomic bombs which detonate at relatively high altitudes.

It is not expected that an atomic bomb would be used to destroy an isolated target by fusing it to detonate at close range. It is recognized, however, that this is a possibility, either by intent or by error.

The characteristics of atomic bombs must be considered in designing "atomic bomb-proof" structures. Unlike the loads used in the design of conventional structures, blast pressure is a transient load, and the mass and vibrational characteristics of the proposed structure must be considered in the design.

If the duration of the impulse is less than one quarter of the period of vibration, the impulse will merely start the swaying of the structure, and the structure will not reach its amplitude of movement until after the cessation of the impulse; the strains occuring when the amplitude is finally reached will be only a fraction of the strains resulting from equal static loads. On the other hand, if the duration of the impulse is several times greater than the period of vibration, the structure will reach its greatest strains or nearly its greatest strains before the impulse is over, and these greatest strains are comparable in magnitude to those resulting from equal static loads. In general, the square of the period of vibration is proportional to the mass and inversely proportional to the stiffness, (so that if the mass is doubled without changing the stiffness, the square of the period will be doubled, and if the stiffness is doubled without changing the mass, the square of the period will be cut in half).

While these premises might lead one to strive for flexibility and mass, it must be remembered that the requirement of thickness for strength works against flexibility. Thickness is also required to shield persons inside.

The individual component units may all have different periods of vibration; all are probably different from the period of the structure as a whole.

The structural engineer's problem is to meet these conditions. One phase of the problem lies in structural dynamics rather than structural statics.

Allowable working stresses may be increased for blast pressure loads.

PROCEDURE OF DESIGN

The planners must decide what structures are to resist attacks by these new special weapons, what forces should be used in designing the structures and what degree of protection is to be afforded. It will obviously not be feasible to design any structure to resist an atomic bomb which detonates at extremely close range.

The structural designer has many problems to solve which do not occur in buildings subject only to static or "equivalent static" loads. It

will probably be necessary first to design the structure to meet the usual requirements plus special conditions imposed by the bomb, and then to analyze the resulting structure to determine whether or not it meets the dynamic conditions. The first design will probably require revisions to satisfy the latter conditions. While this is a relatively laborious task, it is not an impossible one when all the requirements are known.

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Job Problems and Practice

Five cash awards—$50.00, $25.00, and 3 of $10.00 each are to be made for the best contributions to this department in the current volume year—Sept. 1945 to June 1946.

In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved—in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

What Kind of Cement Stucco? (42-174)

A member of the Institute commenting on the fact that very little has been done recently on the subject of "stucco", makes a suggestion that seems highly discussable. Discussion might indicate what lines might be followed up in new study of stucco. Here is his suggestion: "It has been our experience that a masonry cement of the type now marketed by a considerable number of cement manufacturers offers a better stucco material than straight portland cement. These masonry cements contain roughly 50 per cent of blending material, usually ground limestone, together with air-entraining agents. Both experience and theory indicate that such cements will give more satisfactory results than the excessively strong, rigid, and brittle, straight cement mortars".

Setting Heavy Machinery on Concrete Bases* (42-175)

by R. R. KAUFMAN†

The discussions of the subject of grouting machinery bases in the February 1945 and June 1945 issues of the Journal have been read with great interest. The need for improved grouting methods has long been recognized. That there are wide differences in grouting requirements which vary depending on loads, vibration, impact and whether the grout

*See also contributions on same subject (41-164) in February and June 1945 JPP sections.
†Chief Engr., The Master Builders Co., Cleveland, O.

will be subject to oil, water, heat, or other conditions which can affect its stability, is also known.

Different grouting operations have employed materials such as: Sulfur, lead and sulfur, lead and antimony, cement, sand and aluminum powder, and magnesium oxychloride cements.

Sulfur, lead and sulfur combined, and lead and antimony combined, have an advantage in that they may be melted, poured in place, and will harden immediately upon cooling. The lead-sulfur and lead-antimony combinations produce "shrinkproof" grouts. Sulfur alone or in combination with lead fractures rather easily under impact. Lead-antimony will withstand considerable impact. However, because it is ductile it tends to flatten out and creep under continued vibration and impact, and does not recover or expand again to take up the space caused by such impact.

The above materials are usually employed only in minor operations because of (1) difficulty in the case of the sulfur compounds to heat them without burning, (2) danger to the workman in handling the hot molten material and danger of spattering if it comes in contact with water, (3) surrounding concrete must be dry and (4) only small amounts may be poured at a time, otherwise there is danger of disrupting the adjacent concrete by driving off water of hydration.

Aluminum powder is sometimes employed in portland cement grouts to produce expansion by its reaction with the alkali content of the cement which produces hydrogen gas.

This type of grout may be properly designed and controlled by a technician and is satisfactory for grouting equipment with relatively low loadings not subjected to vibration or impact.

Magnesium oxychloride cements have been employed on a minor scale. They have the advantages of acquiring high strengths and of expanding, and the disadvantages of being brittle and soluble in water.

Rust joint iron

Many railroads have found that under bridge seats, turntables, etc. they cannot procure sufficient strength and durability in the time available from grouts made with cement and sand, either poured or packed dry. Therefore, they employ what is termed "rust joint iron" for emergency use in these areas. Rust joint iron was originally prepared by some railroads by mixing cast iron borings from their machine shops with small amounts of ammonium chloride and sulfur. A small amount of water was mixed with this material which was then caulked in place. This produced an expanding, highly ductile grout which could carry loads immediately, and gave satisfactory service under loads, vibration and impact. The difficulties in preparing this type of grout are:

(1) Clean, graded iron borings with low or zero oil content are hard to procure. If too much oil is present the rate of oxidation is slowed to such an extent that failure is likely.

(2) This type of material requires the grout to be under compression at all times. Since in bridge seats and similar grouting work the edges of the grout are unrestrained, the continued oxidation of the unrestrained surface permits ravelling and ultimately causes failure.

(3) Rust joint iron must be caulked in place to be effective as a non-shrink, ductile grout capable of withstanding high loads. This requires competent, semi-skilled labor, and increases supervision, and initial cost as compared with grouting with flowable mixes.

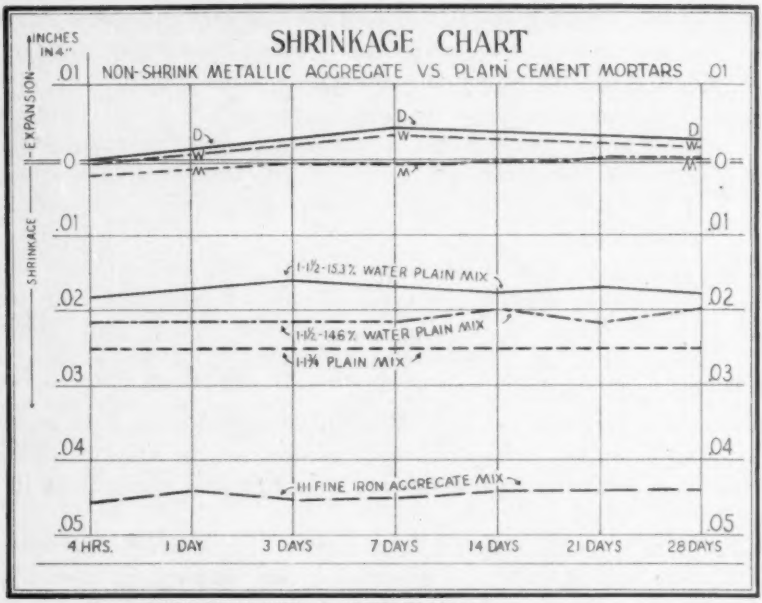
Home-made preparations have about disappeared since most railroads have found it more economical and convenient to employ carefully prepared rust joint iron manufactured by companies specializing in this field.

About 1925 the engineer of a large steel mill employed some metallic floor hardner instead of sand in a cement grout, under some narrow bedplates, and found that it produced in the grout a ductility which greatly increased its resistance to the impact of strip mill operations. This led to the development of an iron aggregate of proper size and grading for grout, combined with materials to promote oxidation and expansion of the metallic particles, and later with water-reducing agents to allow placement with lowered water cement ratio.

In practice, the metallic aggregate is mixed with portland cement and aggregates to a flowable consistency. The grout is then flowed under or around the equipment being grouted, without caulking. Properly used it produces a non-shrink, ductile grout, of high strength to resist vibration and impact. Successful elimination of shrinkage depends in part on the proper oxidation and expansion of the metal particles, and in part on the water reducing agent, which cuts the amount of mixing water required.

This grouting procedure has proved to be practical and economical in large and difficult operations. The service record over 15 years shows no work being replaced due to grout failure.

Non-shrink metallic aggregate should be distinguished from metallic waterproofing or finely divided iron when used for grouting purposes. Finely divided iron, having considerable more surface area, requires an increased quantity of water to maintain workability, which in turn causes additional settlement or shrinkage. This is illustrated in the curves on shrinkage made by Professors Voss and Adams of Massachusetts Institute of Technology, shown in Fig. 1.



D —— 1-1 NON-SHRINK METALLIC AGGREGATE DRY MIX
M - - - 1-1 " " " " MEDIUM MIX
W - - - 1-1 " " " " WET MIX

Fig. 1—Shrinkage chart

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Design of dowel bars in road and runaway joints

J. N. McFeeters and E. T. Hanrahan, *Concrete and Constructional Engineering*, V. 40, No. 2 (Feb. 1946) pp. 27-39 Reviewed by Glenn Murphy

The analysis presented in this paper is based on Professor H. M. Westengaard's treatment of groups of rigid dowels, but is extended to include Mr. B. F. Friberg's analysis of single flexible dowels. Examples are worked out and practical considerations discussed.

Allowance for inclined compression in reinforced concrete beams

A. J. Ashdown, *Concrete and Constructional Engineering*, V. 41, No. 3, (Mar. 1946) pp. 75-81
Reviewed by Glenn Murphy

The author presents formulas for the design of shear reinforcement in beams. The "treatment is based on Dr. Oscar Faber's work on the theory of inclined compression." The results contain simplifying assumptions, such as the neutral axis being located at 0.39 of the depth from the top.

Soil movements as affecting paved surfaces

W. H. Elgar: Surveyor, London, 1944, 103 (2746), 427-8; Publ. Wks., N. Y., 1944, 75 (12), 17-8, 46-7. *Road Abstracts*, Vol. XIII, No. 2, February 5, 1946 Highway Research Abstracts

Soil movements causing cracking of road surfacings parallel to the edge of the road are discussed. Fluctuations in moisture content under the haunches cause expansion and contraction of the subgrade. The moisture content under the center of the road remains fairly constant and hence volume changes are small. Frost heaving, in the author's experience, is more severe in gravel than in other subsoils. The drying and shrinkage of the subgrade under the edges of a road caused by moisture being drawn out of the surrounding soil by vegetation is stated to be the cause of longitudinal cracking of a road during a drought. Methods of preventing cracking due to the above causes are discussed.

The reconstruction of the Longeray Viaduct

Leslie Turner, *Concrete and Constructional Engineering*, Vol. 41, No. 3, (Mar. 46) pp. 63-68
Reviewed by Glenn Murphy

The largest work undertaken in France since 1940 is the reconstruction of the Longeray single track railway viaduct across the Rhone about 15 miles from Geneva. The new reinforced concrete structure, designed by Société des Entreprises Limousin in conjunction with the railway, consists of three parabolic arches with an approach of three semi-circular arches to match the portion of the bridge which was not destroyed in 1940. The outer arch has a span of 208 ft. and a rise of 188 ft., is hollow, and has a

rectangular cross section. Maxium design stress in the concrete was 1060 psi. Forms for each half arch were constructed vertically at the piers and rotated at the springing until they met at the crown. A total of 13,000 cu. yd. of concrete and 550 tons of steel were used in the construction.

Responsibility for defective construction

G. MAGNEL, *Concrete and Constructional Engineering*, Vol. 41, No. 3, (Mar. 46) pp. 69-73
Reviewed by GLENN MURPHY

Professor Magnel of the University of Ghent describes a building insurance plan which has been in operation in Belgium for 10 years. The program covers construction defects and risks of damage to adjacent structures for a 10-year period. The organization, Bureau S. E. C. O., is a cooperative body of organizations of contractors, consulting engineers, and architects, and functions under a technical council of ten. The director is a civil engineer. The total coverage of the Bureau is about $50,000,000 and premiums are about 1.2 per cent. While the organization does not function as a consulting group, it has in effect become an advisory group to most of the civil engineering construction in Belgium. The author states that the scheme has proved successful in Belgium.

Discussion of the plan by Sir Alexander Gibb, Dr. Oscar Faber, and A. E. Butland with references to adoption in Great Britain are included.

The influence of aggregates type and of age on the thermal-expansion of concrete

F. KEIL and H. G. TEMPEL: *Zement*, 1940, 29 (20) 248-53, Road Abstracts, Vol. XIII, No. 3, Mar. 5, 1946
HIGHWAY RESEARCH ABSTRACTS

In the laboratory experiments described, the thermal expansion was measured of concretes made with different types of aggregates, at temperatures of about 5 to 45C. All concretes had a consistency suitable for road surfacing, and a cement content of 560 to 590 lb. per cu. yd. (330 to 350 kg/m^3). The principal results were as follows: (1) The expansion was greatest for Rhine gravel, lump slag and Piesberg sandstone, and least for limestone. (2) The expansion of concrete made with gravelly aggregate was very close in value to figures quoted for the expansion of gravel alone; as the expansion of the aggregate itself decreased, the concrete expansion also decreased, but not in proportion. (3) The expansions at 7 and 28 days were appreciably less than at 90 days. (4) When curing takes place under water, the expansion of the aggregate appears to determine the expansion of the concrete. When curing takes place in air the percentage of air voids and the setting process are also important.

Prestressed concrete

G. MAGNEL, *Concrete and Constructional Engineering*, V. 41, No. 1, (Jan. 1946) pp. 10-20
Reviewed by GLENN MURPHY

In this article of the series Prof. Magnel describes some of the prestressed concrete structures which have been built in Belgium under the auspices of the Laboratory of Reinforced Concrete at the University of Ghent.

A six-track railway bridge of deck construction was built with a different system for each track. Each deck had a span of about 66 ft. and a width of 12 ft. Two of the decks which were reinforced with prestressed wire were 3 ft. 7½ in. thick and each required 119 cu. yd. of concrete as compared with 190 cu. yd. of concrete for a conventional reinforced concrete span with a thickness of 6 ft. 1 in. The weights of steel were 5.7 and 26 tons, respectively.

Other structures built using prestressed concrete include a road bridge, footbridges, pipeline bridge, silos, and an arch roof for a factory. Plans are also being developed for replacing the four supporting piers for the main tower of the St. Nicholas Church in Ghent.

Railway roadbeds stabilized with portland cement grout

ARTHUR J. BOASE, *Civil Engineering* Vol. 15, No. 10 (Oct., 1945) pp. 447-449 Reviewed by J. R. SHANK

Passing trains develop pumping action under track when the ballast and sub-grade are wet and where impervious layers exist on the sub-grade and are developed in the lower strata of the sub-ballast by reason of imperviousness in the original ballast or by penetration of clay from earlier pumping action. Cost of maintenance has been very high in regions where these conditions exist.

Pressure placing of portland cement grout has served to fill these pump chambers with stable material and to seal off inlets and outlets of the same, resulting in greatly reduced costs of maintenance and operation of trains.

Pneumatic pressures and occasionally hydraulic pressures, rarely over 100 psi, serve to inject grouts carrying up to 200 lbs. of sand to the bag of cement through pipe injection points driven to the pocket previously called a pump chamber. Thin neat cement grout is used at the start which is followed by more and more sand until the pocket is filled. The mix and condition of the grout is largely determined by facility for placing as the cost of the cement is less than 20 per cent of the cost of the work.

This work started in 1941 and is now being used by 28 major railroads. On the New York Central lines west of Pittsburgh in 1942, 31 spots were treated. Before treatment these spots were costing $1,187.10 per month in maintenance. Since treatment the cost has been $73.95. The total cost of treatment was $4,334.11.

Aqueduct for Salt Lake City

L. R. DUNKLEY, *Civil Engineering*, Vol. 16, No. 4, pp. 160-162, (April, 1946) Reviewed by J. R. SHANK

Twenty-two and one-half miles of reinforced concrete pipe is part of the aqueduct which connects Deer Creek Reservoir in Provo Canyon, Utah to Salt Lake City, a total distance of 41 mi.

Bell and spigot sections 69 in. diameter by 20 ft. long having 7½ in. walls were constructed and laid. The steel was welded prefabricated. Three sizes of aggregate, sand, ⅜ to ¾, and ¾ to 1½ in. were used. The basic mix was 1:2.25; 3.35 of equal parts of the two coarse aggregate sizes. The concrete was pumped to the forms and external vibrators were used in placing. On completion of the pour the pipe was covered with wooden covers and steam cured at 110 to 130 F. A strength of 750 psi was attained before the forms were removed. This took from 8 to 12 hrs. Steam curing was continued until the strength was 3500 psi. The 28-day strength was 4500 to 5000 psi. Pipe units for bends in alignment were cast with the required bevels in the bells.

A circular rubber gasket was used at the joints. Each successive length was pulled into place by a portable hand winch, installed in the previously laid pipe. Green soap with a vegetable oil base was used as a lubricant. Outside grouting and inside calking completed the joint.

Tests for leakage indicated losses over one 10-mi. length of 235 gal. per mi. in 24 hrs. One 1.7 mi. section of this showed 500 gal. The maximum permitted was 3500 gal. per mi. per 24 hrs.

Measuring the consistency of concrete mixtures

From "*Investigations as to the Consistency of Concrete Mixtures*", N. V. MATERN and N. ODEMARK. Bull. 68, *Statens Vaginstitut*, Stockholm, Sweden, 1944 HIGHWAY RESEARCH ABSTRACTS

The Road Institute of Sweden has carried out a series of investigations with different concrete mixes, trying several methods of measuring the consistency, namely: (1) the flow table, (2) the slump test, (3) the Swedish Vebe-apparatus and (4) the same apparatus equipped with a sinker.

The Vebe-apparatus, Fig. 1, consists of a plane table (A) vibrating 3000 vibrations per minute. To the table is attached a cylindrical vessel (B) 24 cm. in. diameter and 20 cm. high. The pivoting rod (D) carries a circular glass plate (E) which can be turned over the vessel, and made free to move vertically. A form, shaped like a cone (the same as that which is used in the slump test) is placed in the center of vessel (B) and filled with concrete. After removal of the form, the glass plate is turned over the concrete and carefully lowered to contact. The vibrating device is then started. The time is measured from this moment until the concrete is deformed to full contact with the whole underside of the glass plate. The consistency is defined by this time in seconds and is designated °VB.

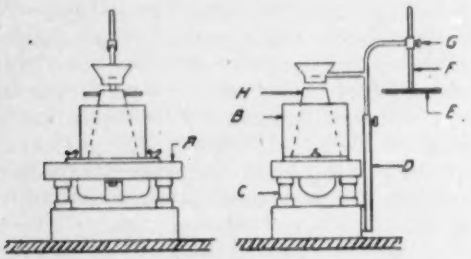

Fig. 1—Vebe-apparatus for measuring the consistency of concrete mixes

The Road Institute has also tried a modification of the measuring device in the Vebe-apparatus. The glass plate was replaced by a sinker (a steel plate fastened to a rod). The consistency was defined as time in seconds for 5 cm. sinking. The consistency thus obtained is called °VB_m.

The flow table gives exact and, with repeated tests, equal values. The method however, can only be used for rather loose consistencies, less than 4 to 5 °VB. The slump test covers about the same range of consistency as the flow table. This test is not very accurate for lean mixes and mixes with a deficiency of the finest particles. The values of repeated tests often have great dispersion. The Vebe apparatus generally gives good values between 2—3 °VB to 25—30 °VB and the dispersion is low. Thus the apparatus is especially useful for rather stiff mixtures which are well compressed at the placing, for instance vibrated concrete for pavements. The method based on using a sinker in the Vebe-apparatus is suitable for mixtures with maximum size of coarse aggregates less than 0.79 to 1.18 in.

Concreting at very low temperatures

The Surveyor and Municipal and County Engineer, Vol. CIV, No. 2812, Dec. 14, 1945, (London).
HIGHWAY RESEARCH ABSTRACTS

The fact that, under extreme conditions of cold, electric current is employed for keeping concrete warm while construction is proceeding is bound to interest engineers, and they will find an account of such and other practices in the U.S.S.R. in a paper entitled "Electro-Concrete," by Mr. K. Billig, published in abridged form in the October, 1945, number of the Journal of the Institution of Civil Engineers (England).

Mr. Billig describes methods normally followed in that region, for keeping concrete from freezing. Temporary hutting is erected, the air inside being heated in various ways. Sand and gravel, frozen hard in transit, are thawed under canvas in large heaps on floors on which a number of lines of perforated steam pipes are laid. The mixing water is heated to about 100-120 F. Concrete mixed at remote plants is conveyed in well insulated containers so that the material cast in the moulds is never at a temperature less than 41 F.

The principle on which heating by electricity is based is that wet green concrete is, to a certain extent, electro-conductive, the resistance being sufficient to raise the temperature considerably, which may be useful in accelerating setting and hardening of the concrete or in preventing it from freezing at too early a stage. Current is passed through the green concrete by introducing two or more poles. Only alternating current can be used, because direct current decomposes the water by electrolysis. It has been found that the leaner the mix and the smaller the proportion of sand to large aggregate, the higher is the electro-resistivity of the concrete; that, for equal mix and consistency, gravel concrete has a higher resistivity than broken-stone concrete and that for the same nominal mix and for similar aggregates a moist concrete has a higher resistivity than a wet concrete. The cost of treatment for a work of about 1,300 cu. yd. of concrete, at a rate of 26 cu. yd. per day, averages 25 to 30 per cent of the contract price. In the case of the construction of a large factory, in the course of which the author supervised the electric warming of 3,270 cu. yd. of concrete, the costs, per cu. yd. in English equivalents, were: current 6s, 5d., ($1.28), other items, 9s, 7d., ($1.92), including labour 3s. 11d. ($0.79) and organization 2s. 5d. ($0.48). At temperatures above 15 F costs would probably be considerably less.

Soil cement in aerodrome construction

W. C. Andrews, *Journal, Engineers Australia* Vol. 17, No. 9 (Sept., 1945) pp. 165-172
Reviewed by J. R. Shank

The construction of two aerodome projects in western New South Wales involved 350,000 sq. yd. of soil cement. The total thickness was 6 in. resting on a compacted sub-base 6 in. thick.

A soil map was prepared in accordance with A.S.T.M. and U. S. Public Road Administration test procedures. The cement content was 10 per cent by volume with a surface enrichment of 4 per cent additional cement. The soils were of a brown, desert type containing lime marl. They ranged from lighty sandy loams of A-2 P.R.A. classification, having a maximum density up to about 130 lb. per cu. ft. at optimum moisture contents ranging from 8 to 12 per cent, and with less than 15 per cent passing the 200 screen, down to light clay loams of A4-A6 type, with maximum densities of about 110 lb. per cu. ft.

Eighty per cent of the soil was specified to pass ¼-in. screen. The moisture content, below which the cement did not deleteriously prehydrate in the dry mix, appeared to be about 9 per cent. It was found that pulverization procedures were most efficiently carried out if this moisture content was brought up to 9 per cent.

On one of the projects all mixing was carried out by windrowing and on the other by rotary mixing. Cement in amounts up to 300 tons per day were spotted in bags over the area a half day in advance. The bags were emptied and spread over the surface to uniform depth with rakes. Dry mixing was most efficiently carried out by rotary hoes making two passes traveling at from 1 to 1½ mph mixing a depth of 4 to 5 ft. Water was added by power and gravity sprayers. Spring-tooth cultivators at maximum tractor speeds or disc harrows followed to disperse the water through the mixtures. Continual traversing of the mix by six-furrow plows assisted in turning over the lower dry material. This wet mixing procedure had to take place within one hour. Rolling operations, every third pass of which was followed by disc harrows to break up any clods formed by the rolling, to within 1½ in. of finished grade took place during the next 1¼ hr. The additional cement used for the top enrichment was spread from trucks during the final stages of sheepsfoot rolling, while water to hydrate this additional cement was added. Finishing then proceeded by grading and rolling with pneumatic rollers, followed first by nail drags to insure against formation of compaction planes

near the surface, and then as compaction approximated to the surface, by final grading, pneumatic rolling, and broom dragging.

Curing because of the drying winds prevalent was very difficult. Sisalkraft blankets about 26 ft. wide by 50 ft. long were applied generally within one hour of finishing. Considerable amounts of soil were placed on the edges and joints as well as here and there on the blankets to hold them down against the wind. At the end of seven days condensation on the under sides of the blankets usually appeared.

The results were a uniform gray close-textured material devoid of streaks and soil nodules without apparent cracking during the curing operations. Cracks appeared after exposure at intervals up to 20 ft. Subsequently, cracks developed at closer intervals but were confined to the surface layers. The surface enrichment tended to form separate laminations of material.

The average cost per sq. yd. was 5 s, 4 d.

The strengths attained were 440 and 660 psi for windrowing and rotary mixing procedures respectively.

Rapid determination of the bitumen content in pavements

HARRY ARNFELT. *Statens Vaginstitut*, Stockholm, Sweden, 1942

HIGHWAY RESEARCH ABSTRACTS

A method for the rapid determination of the tar content of tar mineral masses, such as tar-concrete, has been devised. Rapidity in the process of extraction has been effected by means of a mechanical device, consisting essentially of a cylindrical container, which is rotated about its axis. The direction of rotation is reversed at regular intervals, the extraction liquid thus being made to flow through the bituminous mass. Benzene was used as extracting liquid. The chief advantage of this method of extraction is that the mineral particles are gently rubbed against each other. When tar is dissolved from the surface of the aggregates, it leaves a layer of insoluble organic matter, which prevents the tar underneath this layer from being dissolved. The rubbing of the particles against each other removes the insoluble organic matter. The tar is thus rapidly dissolved, the insoluble organic matter and fine mineral particles being dispersed in the solution. This solution is filtered through a paper filter in a metal funnel. After drying, the material on the filter is removed from the paper and ignited in air, under specified conditions ensuring the non-decomposition of the mineral matter.

The procedure described may be expected to give the percentage of tar in the specimens, within a comparatively short time, i.e., less than 3 hr.

Some sources of error in the determination of the tar content are discussed. In order to test the correctness of the results obtained by this method, some determinations of the tar content of known specimens have been made. The result proved to be satisfactory.

It was considered of some interest to know if this procedure also could be applied to pavements containing asphaltic bitumen. Experiments showed, that extraction was easily performed in the aforesaid extraction apparatus, but the filtration of the solutions of asphaltic bitumen presented some difficulties.

An investigation on the filtration properties of such solutions was therefore carried out.

Even in the case of particle-free solutions of asphaltic bitumen, clogging of the filter often occurs when a hard filter is used. This is due to small amounts of a substance associated with the asphaltic bitumen, the nature of which is still obscure, but which very likely consists of almost insoluble high-polymeric hydrocarbons, related to, but not identical with the carbenes. Apparently this substance gradually renders the filler cake forming on the filter impenetrable to the remaining parts of the asphaltic solutions.

Different solvents were examined regarding their capacity for dissolving asphaltic bitumen and yielding solutions easy to filter. Among the substances tried, a B-dichloro-ethylene was found to be best suited to the purpose.

Filtration became easier when care was taken to suppress the formation of a filler cake, and by use of filters, in which the filler particles were gradually caught and distributed over a comparatively large filter volume. Such filters, consisting of combinations of paper and cloth filters, commencing with a weak filtering cloth and ending with a hard filter paper, proved to be useful for the filtration of solutions of asphaltic bitumen containing fine mineral particles.

It was thus possible to filter solutions of asphaltic bitumen if an adequate filtering medium was employed.

In order to attain greater rapidity, however, it was found better to dispense with the finest filter paper and instead to use a centrifuge to recover the finest mineral particles.

Finally, a chapter on the decrease in weight by heating mineral matter is added. The accuracy of the determination of tar by the above method is largely dependent on the constancy of the weight of the mineral matter by the ignition for the removal of the organic matter.

The mineral matter consists often to a large extent of calcium carbonate, which is likely to decompose during the ignition. The amount of carbon dioxide given off by the decomposition is dependent on the rate of decomposition at the temperature used. It was found by experiments that less than one per cent by weight of carbon dioxide is given off at temperatures below 550 C. during one hour. In order to obtain additional data a study of the literature on this subject has been made.

A few experiments have also been made to ascertain the amount of water given off from powdered rocks, not containing carbonates, during the ignition. These experiments showed that it generally can be expected that the loss in weight caused by the ignition is less than one per cent.

Damage on concrete pavements by wintertime salt treatment

HARRY ARNFELT. *Statens Vaginstitut*, Stockholm, Sweden, 1943 HIGHWAY RESEARCH ABSTRACTS

In the spring following the exceptionally cold winter of 1940-1941 some damage was observed on the surface of a concrete road near the town of Linkoping in central Sweden. On some stretches a great part of the surface had scaled off in comparatively large flakes.

A request was made to the State road institute to investigate the cause of the damage.

It was found that the damages could possibly have been caused by the winter-time treatment of the road with sand mixed with calcium chloride to remove ice.

In order to test this possibility specimens from the road were compared with specimens taken from another road, in the vicinity of the town of Orebro in central Sweden, which had not been treated with calcium chloride and which was free from damage.

These specimens were then subjected to the influence of aqueous calcium chloride solutions under conditions of alternate freezing and thawing. Preceding the description of the freezing and thawing tests is a chapter discussing the physiochemical equilibrium between a salt and water on the basis of the equilibrium diagrams of sodium chloride-water and calcium chloride-water.

Prior to the tests which the specimens mentioned, some freezing and thawing tests were made on small pieces of concrete chiseled from a concrete pavement. The purpose of these tests was to find the strength of the calcium chloride solution which acted most quickly on concrete.

When submitted to the freezing and thawing tests the specimens were completely immersed in solutions with different percentages of calcium chloride. The vessels containing the solutions were placed in a refrigerator for 12 hours at a temperature of −25 C. (−13 F.). After that time they were taken out and allowed to stand in the laboratory at room temperature for 12 hours. The freezing and the subsequent thawing were repeated in the same manner until the specimens had disintegrated to a required degree.

It was found that a solution of calcium chloride having a concentration of approximately 3.5 per cent had the strongest detrimental effect on the concrete specimens. Weaker as well as stronger salt solutions were far less detrimental.

So far as the author knows it has earlier been assumed that the strongest solution had the strongest action. In fact, those solutions which were too concentrated to freeze even at the lowest temperature used had no observable action at all.

Some freezing and thawing tests were made on specimens from the same source by the use of aqueous solutions of other substances.

Rock salt was found to act most strongly in 5 per cent solution. Sodium chloride, barium chloride, potassium ferrocyanide, potassium ferricyanide, ethyl alcohol, and urea in solutions having the same depression of the freezing point as a 3.5 per cent calcium chloride solution were found to disintegrate concrete strongly. (Except in case of potassium ferrocyanide, owing to the formation of a protecting layer of potassium calcium ferrocyanide).

Strong solutions of ethyl alcohol and urea had a comparatively minor power of disintegration.

For comparison some freezing and thawing tests were carried out on specimens of brick and of sand-stone. These specimens were, in contrast to those of concrete, disintegrated by pure water to the greatest degree. Dilute calcium chloride also affected the specimens but to a lesser degree. Solutions stronger than 5 per cent showed very little action either on brick or on sand-stone.

The most aggressive aqueous solution of calcium chloride, 3.5 per cent, was used for the freezing and thawing tests on the concrete specimens from the Linkoping and Orebro roads. After seven cycles of freezing and thawing, heavy damages were observed. It was thus shown that the damage on the road was probably caused by the detrimental action of dilute calcium chloride solutions, which were formed by calcium chloride and melting ice.

Furthermore, it was shown that a mechanical action on the surface of the concrete, such as blows from a hammer, which in itself had no visible effect, accelerated the disintegration.

In addition, some conclusions of more general interest can be drawn from the freezing and thawing experiments.

Dilute, hypoeutectic, aqueous solutions of different substances have, when freezing, an ability to disintegrate concrete. It has been shown that the power of destruction of solutions for some substances varies with the concentration and reaches a maximum at a certain, not very high, concentration.

Most probably this conclusion can be generalized to include all substances which are sufficiently soluble in water. The fact that the most active solutions always seem to be hypoeutectic, i.e., those solutions, in which at lower temperatures ice is primarily crystallizing, strongly suggests that the disintegrating agent is the ice crystallizing from the solution when the temperature is decreasing in the fine capillary system of the concrete.

ACI NEWS LETTER

Vol. 17 No. 6 JOURNAL of the AMERICAN CONCRETE INSTITUTE June 1946

SYMPOSIUM ON ENTRAINED AIR IN CONCRETE

The session on Air Entrainment at the 42nd Annual Convention in Buffalo contained a group of short contributions, each, in most cases, covering only the high-lights of a more comprehensive paper on the subject.

In this JOURNAL (pp.601 to 700) these fourteen contributions have been "filled out" and, with a short added introduction, are presented as a symposium on this timely subject.

Since the last symposium on Air Entrainment in the June 1944 JOURNAL much has been learned about air entrained in concrete and these papers add much to the published knowledge. Many questions have been answered. Many more are raised. Meters, dispensing devices for introducing admixtures, methods and apparatus for determining air content have been developed and are described. The problems of the designers, producers and users are discussed and the solutions to some of them are described.

The symposium, with discussion, will soon be combined with related papers by C. E. Wuerpel, "Field Use of Cement Containing Vinsol Resin" (Sept. 1945) and "Laboratory Studies of Concrete Containing Air-Entraining Admixtures" (Feb. 1946) into a second publication by the ACI on "Air Entrainment in Concrete."

MEMBERSHIP APPLICATIONS INCREASE

Records of long standing were broken in May when the Secretary's office received 112 applications for membership during the month. This number has not been equaled since the late 1920's.

The names of applicants approved by the Board of Direction will be published in the September JOURNAL.

WHO'S WHO in this ACI JOURNAL

Bryant W. Pocock

whose paper, "Asphaltic Oil-Latex Joint-Sealing Compound," appears on p. 565, attended Wayne University, Detroit; John Hopkins University, Baltimore, Md. (A. B. degree 1927); Ludwig-Maximilians Universität, Munich, Germany; Privatlaboratorium Benders und Hobeins, Munich, Germany; and the University of Michigan, Ann Arbor (M. A. degree 1934).

With the exception of two years during the late war, when he was chief chemist for Nash-Kelvinator's Propeller Division, Mr. Pocock has been employed by the Michigan State Highway Department since 1937, his present position being that of chemical research engineer. For the past four years he has been consulting editor on the staff of Products Finishing magazine and is the author of numerous technical papers published in this and other journals.

Mr. Pocock is a member of the American Chemical Society, the Electrochemical Society, the American Society for Metals, the American Electroplaters' Society, and the Photographers' Society of Michigan.

Roger Rhoades

a member of the ACI since 1942, and co-author of the paper "Petrography of Concrete Aggregate" (p. 581), attended the University of California at Berkeley for both undergraduate and postgraduate work in geology. Following work in Mexico and Venezuela he was for a time Assistant Professor of Geology at Antioch College, Ohio, then in 1934 joined the Tennessee Valley Authority as an engineering geologist. In 1941 he went to the Bureau of Reclamation to organize their Petrographic Laboratory and to participate in research relative to the deleterious reaction of some aggregates with high-alkali cements, where he currently supervises the Bureau's Geology Section including the Petrographic and Earth Materials Laboratories and, in addition to other functions, engages in the search for construction materials, including concrete aggregate.

Richard C. Mielenz

co-author (with Roger Rhoades) of the paper on "Petrography of Concrete Aggregate" (p. 581) attended the University of California (Berkeley) receiving his Ph.D. in 1940. After graduation he was employed as geologist by the Standard Oil Co. of California in the San Joaquin Valley from 1939 to 1941. He then joined the Petrographic Laboratory of the Bureau of Reclamation where he has been engaged in application of petrography and geology to engineering problems, particularly selection of concrete aggregates and riprap materials for use on Bureau projects. In addition, he is conducting research into basic problems concerning construction materials, especially the problem of cement-aggregate reaction.

In April 1946 he published a paper entitled "Petrographic Examination of Concrete Aggregates" in the Bulletin of the Geological Society of America. An article written in collaboration with C. J. Okeson entitled "Foundation Displacements Along the Malheur River Siphon as Effected by Swelling Shales" appeared in Economic Geology for May 1946. Mr. Mielenz joined the Institute in 1945.

W. A. Cordon

whose paper entitled "Entrained Air—A Factor in the Design of Concrete Mixes" appears on p. 605 received a B.S.C.E. degree from Utah State College in 1935, and since graduation he has been continuously in the employ of the U. S. Bureau of Reclamation. For 3 years he was Concrete and Earth Control Engineer on the Upper Snake River Project, Idaho, during construction of Island Park Dam and Cross Cut Canal and Diversion Dam.

He also spent 2½ years as Construction Inspector on the Grand Coulee Dam, Washington. Since 1941 he has been stationed at Denver, Colorado, and at the present time is Head of the Concrete, Aggregate and Mix Investigations Unit of the Materials Laboratories.

L. E. Andrews

regional highway engineer, Portland Cement Association, New York City, is author of "Recent Experiences with Air-Entraining Portland Cement Concrete in the Northeastern States," p. 621. For many years he has taken an active part in the design of concrete mixtures and concrete performance studies and has followed closely all phases in the development of air-entrained concrete since its introduction about 1938.

Prior to going with PCA, Mr. Andrews was district engineer, New Jersey State Highway Dept.; assistant district engineer, Pennsylvania Dept. of Highways and resident engineer, Illinois Division of Highways.

During World War I he served with the Corps of Engineers as Private, Corporal, Sergeant, Second Lieutenant and Captain. He was engaged on military road construction and maintenance work in this country and in France. Mr. Andrews is a graduate in civil engineering from Pennsylvania State College.

Alexander Foster, Jr.

a member of the Institute since 1912 and Director from the Third District is Vice-President of the Warner Company, Philadelphia, Pa. His paper, "Experiences with Air-Entraining Cement in Central-Mixed Concrete" (p. 625) is his second contribution to the JOURNAL.

Stanton Walker and Delmar L. Bloom

are co-authors of "Studies of Concrete Containing Entrained Air" (p. 629).

Mr. Walker, Vice-President of the Institute and a member since 1921 is a frequent contributior and needs no introduction to JOURNAL readers. He is also co-author with W. H. Klein on "A Method for Direct Measurement of Entrained Air in Concrete" (p. 657).

Mr. Bloom is in immediate charge of the laboratory of the National Ready Mixed Concrete Association and of the National Sand and Gravel Association Research Foundation at the University of Maryland where he has been employed since January 1, 1945. While attending Iowa State College, where he graduated in Civil Engineering in 1943, he was employed by the Iowa Engineering Experiment Station. After his graduation he worked for a year for the Chicago Bridge and Iron Company before entering the concrete research field.

Henry L. Kennedy

author of "Homogeneity of Air-Entraining Concrete" (p. 641) is not new to these pages, having been the author of several contributions in recent years. Mr. Kennedy has been a member since 1934 and is a Director-at-Large.

Edward W. Scripture, Jr.

author of "Methods of Entraining Air in Concrete" (p. 645), a member of the Institute since 1931, is Director of Research for the Master Builders Co. Author of "Metallic Aggregate in Concrete Floors," *Proceedings* Vol. 33, and of several discussions, Dr. Scripture needs no further introduction to JOURNAL readers.

A. T. Goldbeck

a member of the Institute since 1926, member of the Publications Committee and author of many papers is well known to the ACI. His paper "Effect of Air Entrainment on Stone Sand Concrete," appears on p. 649.

William H. Klein

co-author with Stanton Walker of a paper entitled "A Method for Direct Measurement of Entrained Air in Concrete" (p. 657), a member of the Institute since 1924

and of the Publications Committee, is Vice-President and General Operating Manager of the Pennsylvania-Dixie Cement Corp. Mr. Klein graduated from the University of Michigan with the degree Bachelor of Science in Chemical Engineering in 1906.

From 1906-11 he was with the United Kansas Portland Cement Co. In 1926 he was with the Dixie Portland Cement Co., when he became General Manager of the Pennsylvania-Dixie Cement Corp. He has held his present position since 1933. Mr. Klein is also a member of American Society of Testing Materials and the American Society for the Advancement of Science.

R. R. Kaufman

member of the Institute since 1937, has spent the past 30 years in the concrete field in both routine and research laboratory work, as well as field experience throughout United States and Canada.

He was with the Sandusky Cement Company from 1916 to 1923, as Chief Chemist of the Cleveland Laboratory.

In 1923 he became affiliated with the Master Builders Company as Chief Chemist, and in 1927 became Chief Engineer, the position he holds at the present time.

During these years he has been engaged in the development of techniques, products and equipment related to concrete work. His paper, "Automatic Dispensing Equipment for Air-Entraining Agents" appears on p. 669.

E. M. Brickett

author of a paper entitled, "Mechanical Dispensing Devices for Air-Entraining Agents" (p. 673), although not a member of the Institute, is not new to ACI readers, having been co-author with J. C. Pearson of a paper "Studies of High Pressure Steam Curing." Mr. Brickett is with the Cement Division of the Dewey and Almy Chemical Co.

S. W. Benham

whose paper "A Simple Accurate Method for Determining Entrained Air in Fresh Concrete" (p. 677) is his second contribution to ACI literature, is Research Engineer for the State Highway Commission of Indiana, Bureau of Materials and Tests.

W. F. Kellermann

an ACI member since 1938 and a frequent contributor to professional journals, needs no introduction on the occasion of his paper "Effect of Use of Blended Cements and Vinsol Resin-Treated Cements on Durability of Concrete" (p. 681).

W. H. Herman

author of a paper on "Air-Entraining Concrete" (p. 689) has been employed by the Pennsylvania Department of Highways for the past 21 years. In 1944 he was appointed to his present position as Chief Research Engineer, supervising the activities of the State Testing Laboratory and the Planning, Traffic, Architectural and Forestry Units.

Bertrand H. Wait

member of the Institute since 1939, graduated from Cornell University, C. E. degree in 1902. He spent several years on New York Subways; five years with Board of Water Supply on Catskill Aqueduct work and as Division Engineer on city tunnels when he resigned in 1913. From 1913-1918 as Division Engineer in charge of eleven counties in New York State he was in charge of Design and Construction of a large percentage of the early concrete roads built by the New York State Highway Dept. From 1918-1929 he was Eastern Manager for the Portland Cement Association in charge of the eleven northeastern states. Since that time he has been actively interested in the development and promotion of construction materials principally in connection with concrete and asphalt pavements in the northeastern states. He is President of The Wait Associates, Inc., and consulting engineer for various construction material manufacturers. He is a member of the A.S.T.M. and A.S.C.E. His paper,

"Portland-Rosendale Cement Blends Give High Frost Resistance," appears on p. 697.

Roderick B. Young

member of the Institute since 1917; author of many papers; member of the Board of Direction as Director, Vice-President, President and Past-President for 16 years; is one of our best known authors. His paper "The Repair of Concrete: An Introduction" appears on p. 701.

E. H. Praeger

an ACI member since 1927 contributes his first paper to the JOURNAL in many years, entitled "Behavior of Concrete Structures Under Atomic Bombing" (p. 703). Capt. Praeger's contribution is based upon observations made as a member of a group of qualified officers and technical experts sent to Japan by the Secretary of the Navy after the war to survey damage wrought by the atomic bombs and other weapons on targets in Japan.

Navy Seeks Former Seabees

Former Seabee Warrant Officers, as well as other qualified men and officers with a background in construction work, are being sought to fill approximately 50 vacancies in the regular U. S. Navy Warrant Officer grades.

Needed particularly for work at overseas naval bases and other projects administered by the Bureau of Yards and Docks, the Warrant Officers, including Chief Warrant Officers, would be assigned to varied duties involving all types of naval shore maintenance and construction.

Former Warrant and Commissioned officers are eligible if they had not passed their 37th birthday on the date their first appointment above Chief Petty Officer was made by the President. Other eligibility requirements are set forth in Bureau of Naval Personnel Circular Letter 288-45 (revised).

Honor Roll

February 1 to May 31, 1946

Anton Rydland heads the list with 6½ new members since February 1, 1946.

Anton Rydland	6½
Charles E. Wuerpel	5½
Jacob Fruchtbaum	5
C. C. Oleson	4
Walter H. Price	4
Birger Arneberg	3
D. R. Cervin	3
C. A. Hughes	3
James J. Pollard	3
F. E. Richart	3
K. E. Whitman	3
H. F. Gonnerman	2½
Karl W. Lemcke	2½
Wm. R. Waugh	2½
A. Amirikian	2
A. J. Boase	2
Aloysius E. Cooke	2
E. F. Harder	2
Dean Peabody	2
Henry Pfisterer	2
Flory J. Tamanini	2
I. L. Tyler	2
H. M. Hadley	1½
John J. Murray	1½
H. C. Shields	1½
Lewis H. Tuthill	1½
Stanton Walker	1½
Julius Adler	1
J. F. Barton	1
F. J. Beardmore	1
Rene L. Bertin	1
R. F. Blanks	1
M. E. Capouch	1
Frank W. Chappell	1
Anthony D. Ciresi	1
Emil W. Colli	1
F. K. Deinboll	1
C. T. Douglass	1
H. B. Emerson	1
Ray C. Giddings	1
Elliot A. Haller	1
Issac Hausman	1
M. J. Hawkins	1
Denis O. Hebold	1
Elmo Higginson	1

W. M. Honour....1
O. W. Irwin....1
W. W. Johnson....1
Wm. R. Kahl....1
R. R. Kaufman....1
Henry L. Kennedy....1
H. Walter Leavitt....1
James E. McClelland....1
F. R. McMillan....1
M. J. McMillan....1
Edward P. McMullin....1
Robert L. Mauchel....1
D. G. Marler....1
Hugh Montgomery....1
Ben Moreell....1
Philip Paolella....1
D. E. Parsons....1
J. C. Pearson....1
A. F. Penny....1
C. J. Posey....1
C. C. Pugh....1
E. M. Rawls....1
John A. Rubling....1
Raymond C. Reese....1
Erik Rettig....1
George P. Rice....1
J. L. Savage....1
G. R. Schneider....1
John C. Seelig....1
Thomas C. Shedd....1
LeRoy A. Staples....1
J. W. Tinkler....1
Bailey Tremper....1
Harold C. Trester....1
Oscar J. Vago....1
David Watstein....1
George C. Wilsnack....1
Ralph E. Winslow....1

The following credits are, in each instance, "50-50" with another Member:

A. Arnstein
Kasim Atlas
Michel Bakhoum
Paul L. Battey
E. Ben-Zvi
R. H. Bogue
J. M. Breen
Ernest L. Brodbeck
Fred Burggraf
Fred Caiola
T. F. Collier
Sam Comess
R. W. Crum
Clifford Dunnells
G. J. Durant
A. C. Eichenlaub
Axel Erikson
Cevdet Erzen
E. E. Evans
H. F. Faulkner
K. P. Ferrell
G. V. Gezelius
H. J. Gilkey
E. A. Glennen
D. E. Gondolfi
Howard A. Gray
Per O. Hallstrom
Shortridge Hardesty
H. L. Henson
H. D. Humphries
Frank H. Jackson
W. D. Kimmel
Wm. Lerch
Douglas McHenry
Eugene Mirabelli
C. C. More
A. B. L. Moser
Fernando Munilla
Wm. T. Neelands
Wm. D. Painter
George P. Palo
R. S. Phillips
Herman G. Protze, Jr.
R. D. Rader
Frank A. Randall
N. L. Shamroy
T. E. Shelburne
C. E. Shevling
Charles M. Spofford
D. J. Steele
John Tucker, Jr.
Jose Vila
Ray V. Warren
W. W. Warzyn
Eugene P. H. Willett
R. B. Young

Clayton N. Ward

member of the Institute since 1945, and John A. Strand announce that they have formed a new firm of consulting engineers under the name of Ward and Strand. The firm will engage in engineering projects involving water, diesel and steam power; water works; sewerage works; flood control; irrigation and drainage; hydraulic research; and hydrological investigation.

S. C. Hollister

Dean of the College of Engineering at Cornell, an Institute member since 1917, former Director and President (1932-33) has been named to the new vice-presidency recently created by the Board of Trustees at Cornell University. Dean Hollister will continue as head of the College of Engineering while assuming his new administrative duties. As vice-president, he will be in charge of University development.

F. E. Richart

member of the Institute since 1917, former Director and Past President (1939), has been elected a member of the Board of Directors of the American Society for Testing Materials for a term of three years.

George R. Wernisch

recently discharged from the Navy as a lieutenant commander has just been appointed manager of Ceco Steel Products Corporation's steel joist and roof deck division. He will make his headquarters at Ceco's general offices and plant No. 1 in Chicago.

New Members

The Board of Direction approved 52 applications for Membership (37 Indidvi-ual, 3 Corporation, 8 Junior, 4 Student) received in March and April as follows:

Antrim, Hugh B., Box 35, Juneau, Alaska.

Altobellis, Julian A., 872 West Peachtree St., Atlanta, Ga.

Banerjee, H. S., 5 Maharsi Devendra Rd., Burra Bazar, Calcutta, India.

Barrada, Baha'Uddin, 201 Engineering Hall, Urbana, Ill.

Bayer, Joseph, 4059 Kendall, Detroit 4, Mich.

Bechtold, Ivan, c/o U. S. E. D. Testing Laboratory, Government Moorings, 8010 N. W. St. Helens Rd., Portland, Ore.

Bjuggren, Ulf, A. B. Betongindustri, Mejerivagen 4, Stockholm 9, Sweden

Columbia Concrete Products Co., 2401 Consaul St., Toledo, Ohio.

Cornell, Holly A., Cornell, Howland, Hayes & Merryfield, Room 5, Smith Bldg., Corvallis, Ore.

Cortland, Abram, 750 Grand Concourse, Bronx 51, N. Y.

Crenshaw, Allen E., 1724 Hyde St., San Francisco, Calif.

Dienst, Frank L., 119 S. Summit St., Bowling Green, Ohio.

Dumaresq, J. Phillip, 73 Pemberton St., Cambridge, Mass.

Earle, M. O., Pittsburgh Testing Laboratory, 421 Canal St., New York 13, N. Y.

Eleta, Fernando, 428 Memorial Drive, Cambridge, Mass.

Elkow, Milton O., 315 Linden Rd., Southgate, Ky.

Ghosn, Raymond S., M. I. T., Cambridge, 39, Mass.

Gonzalez-Rubio, Jr., Elberto, Dept. of Architecture, Georgia School of Technology, Atlanta, Ga.

Hayes, John M., Public Road Administration, 303 Old Post Office Bldg., Little Rock, Ark.

Kelsall, K. J., 47 Waugh St., North Perth, Western Australia.

King, John, 1579 Metropolitan Ave., Bronx 62, N. Y.

Kochanski, Anthony J., Ordnance Works, Weldon Springs, Mo.

Lancashire County Council, County Surveyer & Bridgemaster, County Offices, Preston, Lancashire, England.

Lebelle, Jean, Les Procedes Techniques de Construction, 9 Place des Ternes, Paris 17° France.

Lin, Yuan-ti, U. S. Bureau of Reclamation, Denver 2, Colo.

Lindh, R. T., Twin City Readymix Concrete Co., 2840 Aldrich Ave., S., Minneapolis 8, Minn.

Lowey, Leslie L., 69-61 184th St., Flushing, N. Y.

Lum, Walter, 2021 Cornell Rd., Cleveland 6, Ohio.

Manning, Rear Admiral J. J., Chief, Bureau of Yards & Docks, Navy Dept., Washington 25, D. C.

Mattimore, H. S., 4226 E. Amerton Ave., Colonial Park, Pa.

Mazas, Ing. Enrique, Calle A No. 15, ent. la. y3a, Vedado, Havana, Cuba.

Myott, E. B., 6 Virginia Rd., Reading, Mass.

Naulty, R., c/o Hume Pipe Company (Aust.) Ltd., Upham St., Subiaco, Western Australia.

Nordahl, A., Room 56, N. Y. C. R. R. Bldg., W. Third & St. Clair Ave., Cleveland 4, Ohio.

Pinnell, Richard L., 2375 La Jolla Ave., San Diego 10, Calif.

Powell, W. S., c/o Yonge & Hart, Architects, P. O. Box 928, Pensacola, Fla.

Rahn, George A., Highway Research Board, 2101 Constitution Ave., Washington 25, D. C.

Rivett, J. W., 401 Collins St., Melbourne C. 1, Australia

Schlyter, Ragnar, c/o Statens Hantverksinstitut, Stockholm 4, Sweden.

Schultz, Philo Howard, 118 N. Yankton Ave., Pierre, S. Dakota.

Seeck, Roy H., 441 Manor Pl., N. W., Washington 10, D. C.

Seifried, John F., Ceco Steel Products Corp., 1926 S. 52nd Ave., Chicago 50, Ill.

Solel Boneh Ltd., P. O. Box 563, Haifa, Palestine.

Thadaney, K. V., The Concrete Association of India, P. O. Box 312, Mount Rd., Madras, India

Torregrosa, Miguel, 76 Ponce de Leon Ave., Santurce, Puerto Rico.

Vaughan, J. Philip, 294 North St., Halifax, N. S., Canada.

Whitsett, James J., c/o W. F. Schenck, Trenton, Ohio.

Williams, L. G., 3611 Watson Ave., Toledo 12, Ohio.

Wood, Earle H., 505 Delaware Ave., Buffalo 2, N. Y.

Woodroffe, V. M., 453 Oxford St., Halifax, N. S., Canada.

Woolley, W. R., Federal Works Agency, Public Roads Adm., 2938 E. 92nd St., Chicago 17, Ill.

Yasko, Karel, Box 42, Wausau, Wis.

Joseph L. Duffy

Word has recently been received of the death of Mr. Duffy, April 5, 1946 at the age of 53. He had been a member of the Institute since 1936.

Graduating from Armour Institute in 1915 as a structural engineer, he spent most of his career in Chicago and vicinity engaged in engineering design and construction of highways, bridges and public and private buildings. His most recent work was the construction of a large sanitarium at Kalamazoo, Michigan.

Conde B. McCullough

member of the American Concrete Institute since 1939 died suddenly at his home in Salem, Oregon, on May 6, 1946. Dr. McCullough had been connected with the Oregon State Highway Commission since 1919, first as Bridge Engineer and, since 1937, as Assistant State Highway Engineer. He was fifty nine years old at the time of his death.

He was born in South Dakota and received his engineering training at Iowa State College where he received the degree of Bachelor of Science in 1910 and the professional degree of Civil Engineer in 1916. Prior to his college days, with engineering even then in mind, he served as chainman, rodman and transitman on the Illinois Central Railroad. Following his graduation and after a few months with the Marsh Engineering Company of Des Moines, he joined the Iowa State Highway Commission as designing engineer and later became assistant state highway engineer. Oregon State College brought Dr. McCullough to the Pacific Coast in 1916 as professor of civil engineering, where he remained until 1919 when he became Bridge Engineer for the Oregon State Highway Commission. During his long service in this capacity he supervised the design and construction of many bridges, the most noted being the major structures on the Oregon Coast Highway, well known throughout the nation for their beauty.

In 1936 Dr. McCullough was granted leave by the state and for eighteen months was in charge of design and construction of bridges on the Inter-American Highway with headquarters at San Jose, Costa Rica. On his return to Oregon, he became Assistant State Highway Engineer.

In recognition of his accomplishments and of his service to the state, the Oregon State College conferred on him the degree of Doctor of Engineering in 1928. He also found time to study law and was graduated from Willamette University with the degree of Bachelor of Law in 1928.

Dr. McCullough was the author of numerous papers published by the Institute and several technical books. His latest work, published a few weeks before

his death, is a treatise on engineering law entitled "The Engineer at Law," and is an outstanding contribution in this field. Besides membership in the American Concrete Institute, he was a member of the American Society of Civil Engineers, the Northwest Society of Highway Engineers, Tau Beta Pi, Sigma Tau, and Delta Theta Pi, honorary law fraternity.

William Johnson Henderson

a member of the Institute since 1942, died May 10, 1946. He was born at Glen Gardner, N. J., October 31, 1899.

He received the degree of Bachelor of Science in Civil Engineering from Lafayette College at Easton, Pa., in 1922 and the degree of Civil Engineer in 1926. After his graduation, Mr. Henderson spent three years with the Bethlehem Steel Company. In 1925, he became an instructor in structural engineering at Purdue University and later was promoted, successively, to assistant professor and associate professor.

He was a member of Alpha Chi Rho fraternity, Chi Epsilon, and an Associate Member of the American Society of Civil Engineers.

Professor Henderson was married to Ruth Elizabeth Banghart of Glen Gardner on October 4, 1924. He is survived by his widow, his mother and a brother, Abraham B. Henderson.

Portland Cement Association Creates New Division

The Portland Cement Association, through its President, Frank T. Sheets, has announced the election, effective July 1, of Dr. A. Allan Bates as Vice President to direct the newly created Division of Research and Development.

Dr. Bates is widely known for his accomplishments in the fields of chemistry, chemical engineering, ceramics and metallurgy. Since 1938 he has been Manager of the Chemical, Metallurgical and Ceramic Research Division of the Westinghouse Electric Corporation.

He holds a Bachelor of Arts degree from Ohio Wesleyan University, where he majored in chemistry; Bachelor of Science in metallurgical engineering from Case School of Applied Science; Doctor of Science, "Magna cum laude," from the University of Nancy, France; and an honorary degree of Doctor of Engineering from Stevens Institute of Technology. He is a member of Tau Beta Pi and Sigma Xi.

In commenting on Dr. Bates' election as Vice President for Research and Development and on the Association's expanded program, President Sheets said, "It should be understood that scientific research and product development are not new ventures for the Portland Cement Association because these have been important functions of the Association for over 30 years. Now, with augmented funds and facilities, the present and expanded staff of able scientists and engineers will, under Dr. Bates' leadership, go on to new heights of accomplishment, and thus enhance materially the research and technical service which the Association has rendered in the past to the users of portland cement and concrete."

Central Concrete Laboratory Moved

The Central Concrete Laboratory of the North Atlantic Division, Corps of Engineers, U. S. Army at Mount Vernon, New York is to be deactivated during the month of June 1946. The purpose of the deactivation is to permit the transfer of the key personnel and equipment to Clinton, Mississippi where a new laboratory is to be created as a division of the U. S. Waterways Experiment Station, Vicksburg, Miss. The new laboratory will be known as the Concrete Research Division of the U. S. Waterways Experiment Station and will be headed by Mr. Charles E. Wuerpel, who is now engineer in charge of the Central Concrete Laboratory.

Sources of Equipment, Materials, and Services

A reference list of advertisers who participated in the Fifth Annual Technical Progress Issue of the ACI JOURNAL—the pages indicated will be found in the February 1946 issue and (when it is completed) in V. 42, ACI Proceedings. **Watch for the 6th Annual Technical Progress Section in the February 1947 JOURNAL.**

Concrete Products Plant Equipment

page

Besser Manufacturing Co., 902 46th St., Alpena, Mich. 436
—Concrete products plant equipment, production

Stearns Manufacturing Co., Inc., Adrian, Mich. 409
—Vibration and tamp type block machines, mixers and skip loaders

Construction Equipment and Accessories

Atlas Steel Construction Co., 83 James St., Irvington, N. Y. 425
—Forms for concrete

Blaw-Knox Division of Blaw-Knox Co., Farmers Bank Bldg., Pittsburgh, Pa. 410-11
—Truck mixer loading and bulk cement plants, road building equipment, buckets, batching plants, steel forms

Butler Bin Co., Waukesha, Wis. 452
—Central mix, ready-mix, bulk cement and batching plants, cement handling equipment

Chain Belt Co. of Milwaukee, Milwaukee, Wis. 430-1
—Mixers, pavers, pumps

Electric Tamper & Equipment Co., Ludington, Mich. 416-17
—Concrete vibrators

Flexible Road Joint Machine Co., Warren, Ohio 432
—Pavement joint and joint installers

Fuller Co., Catasauqua, Pa. 434
—Unloading and conveying pulverized materials

Heltzel Steel Form & Iron Co., Warren, Ohio 454-5
—Pavement expansion joint beams

Jaeger Machine Co., The, Columbus, Ohio 418-19
—Concrete paving equipment

C. S. Johnson Co., The, Champaign, Ill. 448
—Mixing plant equipment

Koehring Co., Milwaukee, Wis. 424
—Tilting and non-tilting construction mixers

Mall Tool Co., 7703 So. Chicago Ave., Chicago 19, Ill. 431
—Concrete vibrators

Master Vibrator Co., Dayton 1, Ohio 466-7
—Concrete vibrators

Ransome Machinery Co., Dunellen, N. J. 435
Paving Mixers

Richmond Screw Anchor Co., Inc., 816 Liberty Ave., Brooklyn 8, N. Y. 456
—Planned form work

Viber Co., 726 So. Flower St., Burbank, Calif. 422-3
—Concrete vibrators

Whiteman Manufacturing Company, 3249 Casitas Ave., Los Angeles Calif. 444-5
—Vibrating and finishing equipment

Contractors, Engineers and Special Services

American Concrete Institute, New Center Bldg., Detroit 2, Mich. 465
—Publications about concrete

Kalman Floor Co., Inc., 110 E. 42nd St., New York 17, N. Y. 420-21
—Floor finishing methods

Prepakt Concrete Co., The, and Intrusion-Prepakt, Inc., Union Commerce Bldg., Cleveland 14, Ohio 437-440
—Pressure filled concrete

Raymond Concrete Pile Co., 140 Cedar St., New York 6, N. Y. 414
—Pile foundations

Roberts and Schaefer Co., 307 No. Michigan Ave., Chicago 1, Ill. 427
—Thin shell concrete roofs

Scientific Concrete Service Corp., McLachlen Bldg., Washington, D. C. 426
—Mix controls and records

Vacuum Concrete, Inc., 4210 Sansom St., Philadelphia 4, Pa. 442-3
—Forms and lifters with suction controlled concrete

Materials

Anti-Hydro Waterproofing Company, 265 Badger Ave., Newark N. J. 468-9
—Waterproofing

Calcium Chloride Assn., The, 4145 Penobscot Bldg., Detroit 26, Mich. 420
—Calcium chloride

Concrete Masonry Products Co., 140 W. 65th St., Chicago, Ill. 415
—Non-shrink metallic aggregate

Dewey and Almy Chemical Co., Cambridge 40, Mass. 450-1
—Air-entraining and plasticising agents

Horn Co., A. C., Long Island City 1, N. Y. 412
—Waterproofing

Hunt Process Co., 7012 Stanford Ave., Los Angeles 1, Calif. 449
—Curing compound

Inland Steel Co., The, 38 So. Dearborn St., Chicago 3, Ill. 446-7
—Reinforcing bars

Lone Star Cement Corp., 342 Madison Ave., N. Y. 428-9
—Cement performance data

Master Builders Co., The, Cleveland, Ohio, Toronto, Ont. 457-464
—Cement dispersing and air-entraining agents

Sika Chemical Corp., 37 Gregory Ave., Passaic, N. J. 470-1
—Waterproofings, plasticizer, and densifier

United States Rubber Co., Rockefeller Center, New York 20, N. Y. 433
—Form lining

Testing Equipment

Baldwin Locomotive Works, Philadelphia 42, Pa. 413
—Testing equipment

ACI Standards—1945

(collected in one publication)

Standards of the American Concrete Institute adopted since the inauguration of the current procedures for their consideration and promulgation under the supervision of the Standards Committee are available in a single publication, as reprinted from the Journal of the American Concrete Institute, June 1945, pages 559-704. Each Standard is also available in a separate print. New editions of the collected ACI Standards will be issued as rapidly as is justified by the completion of technical committee work. Some of the present Standards have had some few editorial revisions. They include changes in the substance of the texts as approved by the ACI Conventions which adopted them and as subsequently ratified by letter ballot of the ACI Membership.

Not included are "proposed standards" presented in recent years, nor proposed or ratified Standards prior to 1937. Some of the latter will have thorough review and eventually come before the Institute for further consideration. The book contains

	Pages
Building Regulations for Reinforced Concrete (ACI 318-41)	559-620
Recommended Practice for the Use of Metal Supports for Reinforcement (ACI 319-42)	621-624
Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)	625-650
Recommended Practice for the Design of Concrete Mixes (ACI 613-44)	651-672
Specifications for Concrete Pavements and Bases (ACI 617-44)	673-700
Specification for Cast Stone (ACI 704-44)	701-704

(148 pages in covers $1.50—to ACI Members $1.00)

These Standards are also available in separate prints in covers at 50 cents each (cheaper in quantity and to ACI Members) except that ACI 319-42 and ACI 704-44, not in covers, are 25 cents each.

AMERICAN CONCRETE INSTITUTE
New Center Building, Detroit 2, Michigan, U. S. A.

INDEX

PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

VOLUME 42—1946

FROM JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, Vol. 17 Sept. 1945, to June 1946 and Part 2, December 1946

This in an Index of:

Original contributions to the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE—papers, reports, discussions by subject, title and author.

For the convenience of the reader who is referring to JOURNAL issues rather than to Bound Volumes (in which discussion is assembled following the paper discussed) there is a reference to JOURNAL issue in which discussion appeared. This reference to discussion is by supplementary page numbers. If the last page of a paper is 28, the first page of the discussion, published later is 28 - 1.

Important aubjects are classified and indexed under approximately 30 main headings each one appearing in its proper alphabetical order in bold face capital letters—as for instance, ARCHITECTURAL DESIGN and other subjects classified under this head, are indented. These samples do not apply fully to any single year's index.

In general, key words to important subjects appear in alphabetical order in addition to the general classification—as for instance "Admixtures" and "Aggregates" each referring to MATERIALS AND TESTS under which all allied references appear, indented. Authors' names and original titles of papers appear in bold face type in proper alphabetical sequence with the subjects, with references to their contributions.

Specific data on Beams are so indexed by reference to ENGINEERING DESIGN or TESTS OF MEMBERS AND COMPLETED STRUCTURES thus avoiding an oversight by the searcher of important allied data.

The complete title of each paper is indexed. This title is repeated in full following the name of the author in its proper alphabetical order. Numerous cross references to parts of the subject matter covered, do not always indicate the full scope. This may be found by looking up the item referred to under the name of the author or the title of the paper.

The readiest use may be made of this index by gaining some familiarity with the main classifications as follows:

ARCHITECTURAL DESIGN
BIBLIOGRAPHIES
BREAKWATERS—DOCKS AND SEA WALLS
BRIDGES
BUILDING CODES
BUILDINGS
BUILDING UNITS
CENTRAL MIXING
COMMITTEES
CONSTRUCTION—DETAILS, METHODS AND COSTS
CURING
DAMS
DURABILITY
ENGINEERING DESIGN
FAILURES
FIRES, FIRE TESTS AND FIREPROOFING
LAWS AND PROPERTIES
MASONRY UNITS
MATERIALS AND TESTS
MIXTURES
PAVEMENT
PERMEABILITY
PIPE
PRECAST UNITS
RAILWAY USES
RESEARCH
RESERVOIRS AND TANKS
SEWERS
SPECIFICATIONS
STRUCTURAL MEMBERS
SURFACES
TESTS AND FIELD CONTROL
TESTS OF MEMBERS AND COMPLETED STRUCTURES
VIBRATION
WATER WORKS

A

Abeles, Paul W.—Disc. Shrinkage and plastic flow of prestressed concrete. (Part 2, Dec. '46)....244–1
Acceptance tests for aggregate—supplemented by petrography; Rhodes and Mielenz.... 581
Advance bases—U. S. Navy—aggregate for. Rasmusson.... 541
Aggregates—petrographic studies supplementing standard acceptance tests. Rhoades and Mielenz.... 581
Air-entraining agents—dispensing—automatic equipment for. Kaufman.... 669
—Use in stucco. (JPP 42-174).... 721
—Dispensing—mechanical devices for. Brickett.... 673
—Cement—vinsol resin—compared with plain cement. Wuerpel.... 49
—Concrete—homogeneity of. Kennedy.... 641
—Concrete—Penna. Dept. of Highways. Herman.... 689
—Portland cement—experience in northeast states. Andrews.... 621
Air entrainment—Dispersion of portland cement. Powers.... 117
—Effect on central-mixed concrete. Foster.... 625
—Effect on ready-mixed concrete. Walker and Bloem.... 629
—Effect on stone sand concrete. Goldbeck.... 649
—Factor in mix design. Cordon.... 605
—In concrete—A symposium.... 601
—Influence of mixing water hardness. Wuerpel.... 401
—Laboratory studies. Wuerpel.... 305
—Measurement by Klein air meter. Klein and Walker.... 657
—Measurement of—simple accurate method Indiana State Highway. Benham.... 677
—Methods for and mechanisms of. Scripture 645
—Vinsol resin-treated cements—effect on durability of concrete. Kellermann.... 681
Air Meter—Klein—Method for measuring entrained air. Klein and Walker.... 657
Alkali—aggregate reaction—expansion test. Blanks and Meissner.... 517
Anderson, A. A.—Maintenance and repair of portland cement concrete pavement.... 477
Andrews, L. E.—Recent experiences with air-entraining portland cement concrete in the northeastern states.... 621
An investigation of the strength of welded stirrups in reinforced concrete beams. Moretto, Oreste.... 141
—Disc. Arnal (Part 2, Dec. '46).....164–1
A method for direct measurement of entrained air in concrete. W. H. Klein and Stanton Walker.... 657
Arnal, Eduardo—Disc. An investigation of the strength of welded stirrups in reinforced concrete beams. (Part 2, Dec. '46).... 164–1
A simple accurate method for determining entrained air in fresh concrete. S. W. Benham.... 677
Asphaltic oil-latex joint-sealing compound. Bryant W. Pocock.... 565
—Disc.—Stanton and author. (Part 2, Dec. '46).... 580–1
Atomic bombing—Effect on concrete structures in Japan. E. H. Praeger.... 709
Automatic dispensing equipment for air-entraining agent. R. R. Kaufman.... 669

B

Behavior of concrete structures under atomic bombing. E. H. Praeger.... 709
Benham, S. W.—A simple accurate method for determining entrained air in fresh concrete.... 677
Benscoter, Stanley U.—Floating block theory in structural analysis.... 205
Betts, Clifford A.—Concrete construction in the national forests.... 1

BIBLIOGRAPHIES

Air-entraining admixtures. Wuerpel.... 305
Air entrainment—Factor in design of concrete mixes. Cordon.... 605
Field use of vinsol resin cement. Wuerpel 49
Dispersion of portland cement. Powers, T. C.... 117
Joint fillers—pavement. Pocock.... 565
Petrography of aggregates. Rhoades and Mielenz.... 581
Blanks, R. F. and Meissner, H. S.—The expansion test as a measure of alkali-aggregate reaction.... 517
Blanks, Meissner and Tuthill—Curing concrete with sealing compounds.... 493
Bleeding—Effect of admixtures on. Wuerpel. 305
—Effect of air entrainment. Kennedy.... 641
Blended cements and vinsol resin-treated cements—effect on durability of concrete. Kellermann.... 681
—Portland-Rosendale—High frost resistance. Wait.... 697
Bloem, Delmar L. and Walker, Stanton—Studies of concrete containing entrained air 629
Boswell Clay C. and Giesecke, Albert C.—Maintenance of heavy concrete structures—Minn. Power Light Co. practice.... 277
Brickett, E. M.—Mechanical dispensing devices for air-entraining agents.... 673

BRIDGES

Highway—Maintenance and repair of. Paxson.... 105
—National forest roads. Betts.... 1
Brine—Effect on concrete. (JPP 42-171).... 90

C

Central-mixed concrete—Effect of air-entraining cement. Foster 625
Cobbles—in mass concrete. (JPP 41-164) . . . 89
Concrete at advance bases. I. S. Rasmusson 541
Concrete construction in the national forests—Betts 1

CONSTRUCTION—DETAILS, METHODS AND COSTS

Advance bases—Coral aggregate. Rasmusson 541
Bridge maintenance. Paxson 105
Dam—Maintenance and repair—Fond du Lac dam. Boswell and Giesecke 277
Machinery base — Grouting. Kaufman. (JPP 42-175) 721
Masonry structures—Maintenance of. Kelly and Keatts 289
Miscellaneous structures—National forests. Betts 1
Mixing—Dispensing air-entraining agents—Automatic equipment. Kaufman 669
Mechanical equipment. Brickett 673
Pavement—maintenance and repair. Anderson 477
Repair of concrete—an introduction. R. B. Young 701
Silos—Farm—proposed recommended practice. Comm. 714 261
Trestle—Wooden—concrete surfaced. Roberts 85
Coral aggregate—advance base construction U. S. Navy. Rasmusson 541
Cordon, W. A.—Disc. Entrained air in concrete, A symposium. (Part 2, Dec. '46) 604–1
—Entrained air—a factor in the design of concrete mixes 605

CURING

Sealing compounds. Blanks, Meissner and Tuthill 493
—Disc. Whitesides, Whitman and author. (Part 2, Dec. '46) 512–1

D

DAMS

Fond du Lac—Heavy concrete repair slab. Boswell and Giesecke 277
Mud Mountain—construction trestle. Roberts 85
Small structures—U. S. Forest Service. Betts. 1
Deformation—Effect of drying. Pickett 165
—Effect of shrinkage and plastic flow. Staley and Peabody 229
Derrick stone and cobbles in mass concrete (JPP 42-170) 89
Disintegration—deterioration of bridges—cause of repair and replacement. Paxson . . . 105
Dispensing air-entraining agents—automatic equipment for. Kaufman 669
—Mechanical devices for. Brickett 673
Dispersing agents—air-entrained concrete. Symposium 601
—Methods of air entrainment. Scripture 645
Dispersion—of portland cement. Powers 117
—Air entraining admixtures. Wuerpel 305
Drying—distribution of. Pt. 1. Pickett 165
—Shrinkage stresses in concrete. Pt. 2 361

DURABILITY

Disintegration—destructive agents—an introduction. R. B. Young 701
—Frost action—effect of blended cements on paving B. H. Wait 697

E

Effect of air entrainment on stone sand concrete. A. T. Goldbeck 649
Effect of brine on concrete (JPP 42-141) . . 90
Effect of use of blended cements and vinsol-resin-treated cements on durability of concrete. W. F. Kellermann 681

ENGINEERING DESIGN

Beams and slabs—shrinkage stresses. Pickett. Part I 165
—Shrinkage stresses in concrete. Part 2 . . . 361
Building frames—moment and shear determinations. Goodwin. (JPP 42-173) 402
Buildings—resistant to atomic bombing. E. H. Praeger 709
Floor units—minimum standard requirements. Comm. 711 245
Reaction distribution in bases of structures. Benscoter 205
Reinforced concrete structures—announcement of proposed manual. Comm. 315 . . . 473
Slabs—hinged base—floating block theory. Benscoter 205
Splices—lapped bars in beams. Kluge and Tuma 13
Entrained air—A factor in the design of concrete mixes. W. A. Cordon 605
—Direct measurement by Klein air meter. Klein and Walker 657
—In Concrete, Symposium 601
—Disc. Kellermann, Stanton, Spindel, (Cordon. (Part 2, Dec. '46) 604–1
—Measurement of—simple accurate method. Indiana State Highway. Benham 677
Ernsberger, Fred M. and Wesley G. France—Disc. Should Portland cement be dispersed? (Part 2, Dec. '46) 140–1
Expansion—as a measure of alkali—aggregate reaction. Blanks and Meissner 517
Experiences with air-entraining cement in central-mixed concrete. Alexander Foster, Jr 625

Exposure of concrete to high temperature. (JPP 42-169)..... 88

F

Field use of cement containing vinsol resin. Wuerpel..... 49
Fine ground cements—Effect of air entrainment—Penna. Dept of Highways experiences. Herman..... 689
Fire Protection—Concrete. (JPP 42-169)... 88
Floating block theory in structural analysis. S. U. Benscoter..... 205
Floor units—Precast—proposed minimum requirements. Comm. 711..... 245
Fond du Lac dam—Repair—compared to Thompson dam. Boswell and Giesecke..... 277
Forbrich, L. R.—Disc. Should portland cement be dispersed? (Part 2, Dec. '46)..... 140-1
Foster, Alexander, Jr.—Experiences with air-entraing cement in central-mixed concrete..... 625
France, Wesley G. and Fred M. Ernsberger—Disc. Should portland cement be dispersed? (Part 2, Dec. '46)..... 140-1

G

Giesecke, Albert C. and Boswell, Clay C.—Maintenance of heavy concrete structures—Minn. Power & Light Co. practice..... 277
Goldbeck, A. T.—Effect of air entrainment on stone sand concrete..... 649
Goodwin, W. C.—Locating points along beam axis corresponding to known moments (JPP 42-173)..... 402
Gordon, P. B.—Disc. Radiant heating by reinforced concrete. (Part 2, Dec. '46)..... 516-1
Grouting machinery bases. Kaufman (JPP 42-175)..... 721
Gunite—Shrinkage and plastic flow in prestressed concrete. Staley and Peabody..... 229

H

Hardness of mixing water—Influence on air entrainment. Wuerpel. (JPP 42-172)..... 401
Hawk, C. A., Jr.—Disc. Radiant heating by reinforced concrete. (Part 2, Dec. '46)..... 516-1
Heavy concrete repair slab—Maintenance of Fond du Lac dam. Boswell and Giesecke. 277
Herman, W. H.—Air-entraining concrete. Penna. Dept. of Highways..... 689
High Temperature—Effect on concrete (JPP 42-169)..... 88
Homogeneity of air-entraining concrete. Henry L. Kennedy..... 641

I

Influence of mixing water "hardness" on air entrainment. (JPP 42-172). Chas. E. Wuerpel..... 401
Intrusion—Stabilizing structures. Kelly and Keatts..... 289

J

Jennings, B. H. and Lagaard, M. B.—Disc. Radiant heating by reinforced concrete. (Part 2, Dec. '46). 516-1
Joint fillers—Pavement—asphaltic oil-latex. Pocock..... 565
Joints and cracks—Pavement—maintenance and repair. Anderson..... 477
Joint-sealing compound—Asphaltic oil-latex. Pocock..... 565

K

Kaufman, R. R.—Automatic dispensing equipment for air-entraining agents..... 669
Keatts, B. D. and Kelly, J. W.—Two special methods of restoring and strengthening masonry structures..... 289
Kellermann, W. F.—Disc. Entrained air in concrete. A symposium. (Part 2, Dec. '46)..... 604-1
—Effect of use of blended cements and vinsol resin-treated cements on durability of concrete..... 681
Kelly, J. W. and Keatts, B. D.—Two special methods of restoring and strengthening masonry structures..... 289
Kennedy, Henry L.—Disc. Should portland cement be dispersed? (Part 2, Dec. '46)..... 140-1
Kennedy, Henry L.—Homogeneity of air-entraining concrete..... 641
Klein air meter—Measurement of entrained air. Klein & Walker..... 657
Klein, W. H. and Walker, Stanton—A method for direct measurement of entrained air in concrete..... 657
Kluge, Ralph W. and Tuma, Edward C.—Lapped bar splices in concrete beams..... 13

L

Laboratory studies of concrete containing air-entraining admixtures. Chas. E. Wuerpel..... 305
Lapped bar splices in concrete beams. Kluge & Tuma..... 13

LAWS AND PROPERTIES

Compressive strength—Effect of air entrainment. Walker and Bloem..... 629
Durability—Effect of air entrainment. Scripture..... 645

Homogeneity—Effect of air entrainment. Kennedy 641
Plastic flow—Prestressed concrete. Staley and Peabody 229
Radiation. Nichols 513
Resistance to brine. (JPP 42-171) 90
Resistance to freezing and thawing—Effect of blended cements and Vinsol resin-treated cements. Kellermann 681
Shrinkage—during drying. Pickett. Pt. 1). 165
—Shrinkage stresses in concrete. Pickett. (Pt. 2) 361
—Prestressed concrete. Staley and Peabody. 229
Linear base reaction—Floating block theory in structural analysis. Benscoter 205
Locating points along beam axis corresponding to known moments. (JPP 42-173). W. C. Goodwin 402
Lagaard, M. B. and Jennings, B. H.
—Disc. Radiant heating by reinforced concrete. (Part 2, Dec. '46) 516-1

M

Machinery bases—Grouting. (JPP 42-175). Kaufman 721
Maintenance and repair of concrete bridges on the Oregon highway system. Paxson 105
—Portland cement concrete pavement. A. A. Anderson 477
Maintenance of heavy concrete structures—Minnesota Power & Light Co. practice. Clay C. Boswell and Albert C. Giesecke. 277
—Restoring and strengthening structures. Kelly and Keatts 289
Masonry cements—Blended—use in stucco. (JPP 42-174) 721
Masonry structures — Restoring and strengthening. Kelly and Keatts 289
Mass concrete—Derrick stone and cobbles in. (JPP 42-170) 89

MATERIALS AND TESTS

Admixtures — Air-entraining — laboratory studies. Wuerpel 305
—Alkali—Effect on expansion. Blanks and Meissner 517
Air-entraining agents—Effect on ready-mixed concrete. Walker and Bloem 629
—Air-entraining agents—Effect on stone sand concrete. Goldbeck 649
—Air entrainment—Penna. Dept. of Highways experiences. Herman 689
Aggregates—Alkali reaction—Expansion test as measure. Blanks and Meissner ... 517
—Coral advance base construction. Rasmusson 541
—Petrographic studies. Rhoades and Mielenz 581
—Stone sand—Effect of air entrainment. Goldbeck 649
Alkali—Aggregate reaction. Blanks and Meissner 517
Blended cements and Vinsol resin-cements—Effect on durability of concrete. Kellermann 681
Cement—Air-entraining—Effect on central-mixed concrete. Foster 625
Cements—Blended—Use in stucco. (JPP 42-174) 721
—Portland-Rosendale blends—High frost resistance. B. H. Wait 697
Curing compounds—White pigment. Blanks, Meissner and Tuthill 493
Portland cement—Air-entraining practice in northeastern states. Andrews 621
—Dispersion of. Powers 117
Vinsol resin cement—Compared with plain cement. Wuerpel 49
Mechanical dispensing devices for air-entraining agents. E. M. Brickett 673
Meissner, Blanks and Tuthill—Curing concrete with sealing compounds 493
Meissner, H. S. and Blanks, R. F.—The expansion test as a measure of alkali-aggregate reaction 517
Methods of entraining air in concrete. E. W. Scripture, Jr. 635
Membrane treatment—Curing methods. Blanks, Meissner, and Tuthill 493
Mielenz, R. C. and Roger Rhoades—Petrography of concrete aggregate 581
Mixing water—Hardness—Influence on air entrainment. Wuerpel 401

MIXTURES

Design—Air entrainment as factor. Cordon. 605
—Effect of air-entraining agents. Andrews. 621
Moment distribution — Floating block theory. Benscoter 205
Moretto, Oreste—An investigation of the strength of welded stirrups in reinforced concrete beams 141
—Disc. Arnal. (Part 2, Dec. '46) 164-1
Mudjacking—Pavement—Maintenance and repair. Anderson 477

N

Nichols, John R.—Radiant heating by reinforced concrete 513
Non-skid concrete surfacing for wooden treatles at Mud Mountain Dam. (JPP 42-168). H. H. Roberts 85

O

Oil-latex—Asphaltic joint-sealing compound. Pocock 565

P

Panel heating—radiation. Nichols 513

PAVEMENT

Maintenance and repair. Anderson 477

Paxson, G. S.—Maintenance and repair of concrete bridges on the Oregon highway system..... 105
Peabody, Dean, Jr. and Staley, Howard R.—Shrinkage and plastic flow of prestressed concrete..... 229
Petrography of concrete aggregate. Roger Rhoades and R. C. Mielenz..... 581
Pickett, Gerald—Shrinkage stresses in concrete. (Pt. I)..... 165
—(Pt. II)..... 361

PIPE

Prestressed containing steel cylinder—Test of. Ross..... 37
Plastic flow of prestressed concrete. Staley and Peabody..... 229
—Shrinkage stresses. Pickett. (Pt. II)..... 361
Pocock, Bryant W.—Asphaltic oil-latex joint-sealing compound..... 565
Points of inflection—Determination in building design. Goodwin. (JPP 42-173). 402
Portland-Rosendale cement blends give high frost resistance. B. H. Wait..... 697
Powers, T. C.—Should portland cement be dispersed?..... 117
—Disc. Ernsberger, France, Staley, Scripture, Weymouth, Spindel, Schmid, Forbrich, Kennedy and author. (Part 2, Dec. '46)..... 140-1
Praeger, E. H.—Behavior of concrete structures under atomic bombing..... 709
PRECAST UNITS

Floor sections—minimum standard requirements. Comm. 711..... 245
Precast floor units — Proposed minimum standard requirements. Comm. 711..... 245
Prepacking—Stabilizing structures. Kelly and Keatts..... 289
Prepakt concrete—Stabilizing structures. Kelly and Keatts..... 289
Prestressed concrete—Shrinkage and plastic flow. Staley and Peabody..... 229
—Pipe containing steel cylinder—Test of. Ross..... 37
Proposed manual of standard practice for detailing reinforced concrete structures—Report of Comm. 315..... 473
—minimum standard requirements for precast concrete floor units—Report of Comm. 711..... 245
—recommended practice for the construction of concrete farm silos—Report of Comm. 714..... 261
Pumping of slabs—Pavement maintenance and repair. Anderson..... 477

R

Radiant heating by reinforced concrete. John R. Nichols..... 513
—Disc. Gordon, Vanderweil, Hawk, Lagaard & Jennings, and author. (Suppl. '46)..... 516-1
—Panel heating. Nichols..... 513
Rasmusson, I. S.—Concrete at advance bases..... 541
Reaction distribution — Floating block theory in structure analysis. Benscoter..... 205
Ready-mixed concrete — Effect of air-entraining agent on properties. Walker and Bloem..... 629
Recent experiences with air-entraining portland cement concrete in the northeastern states. L. E. Andrews..... 621
Refractory concrete. (JPP 42-169)..... 88
Repair of bridges—Oregon highway. Paxson. 105
—of concrete—An introduction. R. B. Young..... 701
—of concrete structures—An introduction. R. B. Young..... 701
Report of Committee 315—Proposed manual of standard practice for detailing reinforced concrete structures..... 473
—Committee 711—Proposed minimum standard requirements for precast concrete floor units..... 245
—Committee 714—Proposed recommended practice for the construction of concrete farm silos..... 261
Rhoades, Roger and Mielenz, R. C.—Petrography of concrete aggregate..... 581
Roberts, H. H.—Non-skid concrete surfacing for wooden trestles at Mud Mountain Dam. (JPP 42-168)..... 85
Rosendale—Portland cement blends—give high frost resistance. B. H. Wait..... 697
Ross, Culbertson W.—Tests of prestressed concrete pipes containing a steel cylinder... 37
Rust joint iron—Grouting of railroad structures. Kaufman. (JPP 42-175)..... 721

S

Schmid, Emil—Disc. Should portland cement be dispersed? (Part 2, Dec. '46)..... 140-1
Scripture, E. W., Jr—Disc. Should portland cement be dispersed? (Part 2, Dec. '46)..... 140-1
—Methods of entraining air in concrete..... 645
Sealing compounds — Curing concrete. Blanks, Meissner, and Tuthill..... 493
Seal—Joint—Asphaltic oil-latex compound. Pocock..... 565
Setting heavy machinery on concrete bases—(JPP 42-175). R. R. Kaufman.... 721
Should portland cement be dispersed?—Powers, T. C..... 117
—Disc. Ernsberger & France, Staley, Scripture, Weymouth, Schmid, Kennedy, Spindel, Forbrich, and author. (Part 2, Dec. '46)..... 140-1
Shrinkage—and plastic flow of pre-stressed concrete. Howard R. Staley and Dean Peabody, Jr..... 229

—Disc. P. W. Abeles and author. (Part 2, Dec. '46)..... 240–1
—Free or restrained. Pickett. Shrinkage stresses in concrete. (Pt. II)..... 361
—Stresses in concrete. Gerald Pickett. (Pt. I). 165
—(Pt. II)..... 361
Silos — Farm — Recommended construction practice. Comm. 714..... 261
Slip—Lapped bar splices. Kluge and Tuma.. 13

SPECIFICATIONS

Floor units—precast—proposed minimum standard requirements. Comm. 711..... 245
Joint-sealing compounds—Asphaltic oil-latex —tentative regulation. Pocock..... 565
Sealing compound—White pigment. Blanks, Meissner and Tuthill..... 493
Silos—Farm—Proposed recommended practice for construction. Comm. 714..... 261
Structures—Proposed manual of standard practice for detailing reinforced conc. structures. Comm. 315..... 473
Spindel, M.—Disc. Entrained air in concrete. A symposium. (Part 2, Dec. '46)..... 604–1
—Disc. Should portland cement be dispersed? (Part 2, Dec. '46)..... 140–1
Splices—Lapped bars in beams. Kluge and Tuma..... 13
Stabilizing structures—Intrusion and prepacking. Kelly and Keatts..... 289
Staley, Howard R.—Disc. Should portland cement be dispersed? (Part 2, Dec. '46)..... 140–1
—**and Peabody, Dean, Jr.**—Shrinkage and plastic flow of pre-stressed concrete..... 229
Stanton, Thomas E.—Disc. Asphaltic Oil-latex joint-sealing compound. (Part 2, Dec. '46)..... 580–1
—Disc. Entrained air in concrete. A symposium. (Part 2, Dec. '46)..... 604–1
Steel cylinder reinforcement in preatressed pipe. Ross..... 37
Stirrups—Welded reinforcement—Tests of beams—Welded stirrup—Tests of. Moretto. 141
Stone sand concrete—Effect of air entrainment. Goldbeck..... 649
Stucco—Effect of type of cement. (JPP 42–174)..... 721
Studies of concrete containing entrained air—Stanton Walker and Delmar L. Bloem.. 629
Symposium—Entrained air in concrete..... 601

T

TESTS AND FIELD CONTROL

Air entrainment content—Relation to field control. Andrews..... 621
Control cylinders—Vinsol resin cements. Wuerpel..... 49
Specimens—Mass concrete. (JPP 42–170). 89

TESTS OF MEMBERS AND COMPLETED STRUCTURES

Beams—Lapped bar splices in. Kluge and Tuma..... 13
—Measuring shrinkage stresses. Pickett. (Pt. II)..... 361
—Welded stirrups. Moretto..... 141
Pipe section—Prestressed containing steel cylinder. Ross..... 37
Tests of prestressed concrete pipes containing a steel cylinder—Ross..... 37
Thermal stresses—Relation to drying. (Pt. I)..... 165
—Shrinkage stresses in concrete. (Pt. II). Pickett..... 361
Transit-mixed concrete—Effect of air-entraining cement. Foster..... 625
Trestle—Wooden—Concrete surfaced. Roberts. (JPP 42–168)..... 85
Tuma, Edw. C. and Kluge, R. W.—Lapped bar splices in concrete beams..... 13
Tuthill, Lewis H., Blanks and Meissner—Curing concrete with sealing compounds.... 493
Two special methods of restoring and strengthening masonry structures—J. W. Kelly and B. D. Keatts..... 289

V

Vanderweil, Raymond G.—Disc. Radiant heating by reinforced concrete. (Part 2, Dec. '46)..... 516–1
Vinsol resin—Air-entraining agent—Effect on ready-mixed concrete. Walker and Bloem.. 629
—Effect on stone sand concrete. (Goldbeck). 649
—Air-entraining concrete—Penna. Dept. of Highways experience. Herman..... 689
—Cement—Field use of. Wuerpel..... 49
—Mixes—Comparison with portland-Rosendale blends. B. H. Wait..... 697
—Treated cements and blended cements—Effect on durability of concrete. Kellermann. 681

W

Wait, B. H.—Portland-Rosendale cement blends give high frost resistance..... 697
Walker, Stanton and Bloem, Delmar L.—Studies of concrete containing entrained air. 629
Walker, Stanton and Klein, W. H.—A method for direct measurement of entrained air in concrete..... 657
Warping—Free or restrained. Pickett. Shrinkage stresses in concrete. (Pt. II)..... 361
Welded stirrups in beams—Tests of. Moretto..... 141

Wetting and dispersion of portland cement. Powers..... 117

Weymouth, C. A. G.—Disc. Should portland cement be dispersed? (Part 2, Dec. '46)..... 140–1

What kind of cement stucco? (JPP 42–174). 721

Whitesides, George W.—Disc. Curing concrete with sealing compounds. (Part 2, Dec. '46)..... 512–1

Whitman, P. G.—Disc. Curing concrete with sealing compounds. (Part 2, Dec. '46)..... 512–1

Wuerpel, Charles E.—Field use of cement containing vinsol resin..... 49

—Influence of mixing water "hardness" on air-entrainment. (JPP 42–172)..... 361

—Laboratory studies of concrete containing air-entraining admixtures..... 305

Y

Young, Roderick B.—The repair of concrete: An introduction..... 701

INDEX

Current Reviews

From JOURNAL OF AMERICAN CONCRETE INSTITUTE, Vol. 17, Sept. 1945 to June 1946

For information as to indexing methods see PROCEEDINGS *Index, p. 733*

A

Action of steel in reinforced concrete pavement. (Review) Wooley. *Highway Research Abstracts*..... 562

Admixtures—Effect on amount of mixing water and workability. (Review) Kronsbein. *Zement*..... 96

Aerodrome construction—Soil cement. (Review) Andrews. *Journal of Engineers Australia*..... 729

Aggregate production at Fontana Dam. (Review) McKamey. *Engineering News-Record*. 270

Air content—Control by natural cement. (Review) Lovewell. *Engineering News-Record*. 265

Allowance for inclined compression in reinforced concrete beams. (Review) Ashdown. *Concrete and Constructional Engineering*.... 725

A mobile laboratory for testing concrete. (Review) *Concrete and Constructional Engineering*..... 553

Aqueduct for Salt Lake City. (Review) Dunkley. *Civil Engineering*..... 727

Automatic accelerated freezing and thawing apparatus for concrete. (Review) Wuerpel and Cook. *American Society for Testing Materials Preprint*..... 93

Automatic slurry sampler. (Review) Lintz. *Rock Products*..... 265

B

Barges—Cargo—Precast. (Review) *Concrete and Constructional Engineering*..... 553

Beams—Limiting reinforcement determination. (Review) Gthler, Amos and Friedrich. *Building Science Abstracts*..... 560

Bituminous content in pavements—Rapid determinations. (Review) Arnsfelt. *Statins-Vaginstitut*..... 730

Bleeding of portland cement paste. (Review) Steinour. *Bulletin 4, Research Laboratory, PCA*..... 557

Brazilian concrete building design compared with U. S. practice. (Review) Boase. *Engineering News-Record*..... 273

Bridge foundations. (Review) Fairhurst. *Concrete and Constructional Engineering*..... 95

BRIDGES

Railway—Deck construction—Prestressed. (Review) Magnel. *Concrete and Constructional Engineering*..... 726

—Viaduct—Longeray—reconstruction. (Review) Turner. *Concrete and Constructional Engineering*..... 725

T-beam—Precast units. (Review) *Engineering News-Record*..... 270

Buckling—Prestressed concrete beams. (Review) Magnel. *Concrete and Constructional Engineering*..... 554

BUILDINGS

Multi-story—Precast units. (Review) Lee. *Concrete and Constructional Engineering*.. 554

Specifications—Brazilian compared to U. S. (Review) Boase. *Engineering News-Record*. 273

C

Calcium chloride—Wintertime treatment—Effect on pavements. (Review) Arnfelt. *Statens Vaginstitut*..... 731

Calculation of raw mixes. (Review) Vera, Jr. *Rock Products*..... 265

Chemical processes in the hardening of portland cement. (Review) Hedin. Swedish cement and Concrete Institute at the Royal Tech. University..... 561

Chimney base at Bradford. (Review) Manning. *Concrete and Constructional Engineering*..... 553

Clinker composition—Manufacture of portland cement. (Review) Vera, Jr. *Rock Products* 265

Clinker rings—Formation and prevention. (Review) Gingerich. *Rock Products*..... 95

Coarse aggregate—Evaluating strength of. (Review) Markwick & Shergold. *Journal of the Institution of Civil Engineers*..... 98

Cold weather concreting—Wall construction at the Soo. (Review) *Engineering News-Record*..... 268

Complete buildings constructed with precast reinforced concrete elements. (Review) Loesser. *Beton u. Stahlbetonbau*..... 272

Compression tests on beams. (Review) Diggory. *Concrete and Constructional Engineering* 554

Compressive strength—Apparent—Effect of type of test specimen. (Review) Mather. *American Society for Testing Materials Preprint*..... 93

Concreting at very low temperatures. (Review) *Highway Research Abstracts*..... 728

Consistency—Measuring. (Review) Matern Odemakk. *Statens Vaginstitut*..... 727

CONSTRUCTION—DETAILS, METHODS AND COSTS

Aerodrome—Soil cement. (Review) Andrews. *Journal, Engineers Australia*..... 729

Aqueduct—Bell and spigot sections. (Review) Dunkley. *Civil Engineering*..... 727

Bridge—T-beam—Precast. (Review) *Engineering News-Record*..... 270
Buildings—Defective construction. Insurance against. (Review) Magnel. *Concrete and Constructional Engineering*..... 726
—Precast reinforced elements. (Review) Loesser. *Beton u. Stahlbetonbau*..... 272
Cold weather operations—Electric heating. (Review) *Highway Research Abstracts*... 728
Dam—Fontana—Aggregate production. (Review) McKamey. *Engineering News-Record*..... 270
—Foundation—Grouting. (Review) Harza. *Engineering News-Record*..... 267
—Spillway—Vacuum processing. (Review) Rippon. *Engineering News-Record*..... 270
Filtration plant—Forms—Special steel. (Review) *Engineering News-Record*..... 269
Form lining—Norfolk Dam. (Review) Pittman. *Engineering News-Record*..... 266
Hangar floor and apron. (Review) *Engineering News-Record*..... 266
Pavement—Patching—Shallow. (Review) *The Highway*..... 558
Railway roadbeds—Stabilizing with grout. (Review) Boase. *Civil Engineering*..... 727
Revettment—New method. (Review) Pickett. *Engineering News-Record*..... 266
South American practices. (Review) Boase. *Engineering News-Record*..... 269
Viaduct—Longeray—Reconstruction (Review) Turner. *Concrete and Constructional Engineering*..... 725
Wall—Cold weather concreting. (Review) *Engineering News-Record*..... 268
Construction practices in South America. (Review) Boase. *Engineering News-Record*.... 269
Contribution to the study of the solution of the lime of portland cements and mixed cements with a portland base. (Review) Zollinger. *Zement*..... 556
Creep of concrete of high overload. (Review) Shank. *Engineering Exp. Station News*..... 559
Creep—Prestressed concrete beams. (Review) Magnel. *Concrete and Constructional Engineering*..... 554

D

Damage on concrete pavements by wintertime salt treatment. (Review) Arnfelt. *Statens Vaginstitut*..... 731

DAMS

Fontana—Aggregate production. (Review) McKamey. *Engineering News-Record*..... 270
—Structural behavior revealed by dual-purpose instruments. (Review) Waugh. *Engineering News-Record*..... 268
Norfolk—Form linings. (Review) Pittman. *Engineering News-Record*..... 266
Rincon del Bonete—Uruguay—Grouting foundation. (Review) Harza. *Engineering News-Record*..... 267
Deck bridge built of precast T-beam units. (Review) *Engineering News-Record*..... 270
Design of dowel bars in road and runway joints. (Review) McFeeters and Hanrahan. *Concrete and Constructional Engineering*..... 725
Deterioration—Weathering—Bridge concrete. (Review) Jackson. *Public Roads*..... 99
Disintegration of bridge concrete in the west. (Review) Jackson. *Public Roads*..... 99
Dispersion of portland cement by adsorption of calcium lignosulfonate. (Review) Ernsberger and France. *Industrial and Engineering Chemistry*..... 93
Dowel bars—Design—In road and runway joints. (Review) McPheeters and Hanrahan. *Concrete and Construction Engineering*..... 725

DURABILITY

Disintegration—Weathering—Bridge concrete. (Review) Jackson. *Public Roads*.... 99

E

Effect of heat on portland cements containing Vinsol resin. (Review) *Technical News Bulletin. Bureau of Standards*..... 268
—of length on the strength of compression test specimens. (Review) Tucker, Jr. *American Society for Testing Materials*..... 94
—of type of test specimen on apparent compressive strength of concrete. (Review) Mather. *American Society for Testing Materials*... 93
Electro-concrete—Heating in cold weather operations. (Review) *Highway Research Abstracts*..... 728

ENGINEERING DESIGN

Beams—Shear reinforcement. (Review) Ashdown. *Concrete and Constructional Engineering*..... 725
—Stresses near supports. (Review) Forbrich. *Rock Products*..... 267
Bridge—Arch—Fixed-end. (Review) *Concrete and Constructional Engineering*..... 553
—deck—Prestressed concrete. (Review) Magnel. *Concrete and Constructional Engineering*..... 726
Buildings—Brazilian compared to U. S. (Review) Boase. *Engineering News-Record*..... 273
Chimney base. (Review) Manning. *Concrete and Constructional Engineering*..... 553
Foundations—Bridge. (Review) Fairhurst. *Concrete and Constructional Engineering*... 95
Hangar floor and apron. (Review) *Engineering News-Record*..... 266
Pavement—Rigid type—Joint spacing. (Review) Clemmer. *Highway Research Abstracts*..... 273
Rigid frames. (Review) Hilal. *Costa Tsoumas and Co. Press*..... 95
Road and runway joints. (Review) McFeeters and Hanrahan. *Concrete and Constructional Engineering*..... 725

Viaduct—Arch—Longeray. (Review) Turner. *Concrete and Constructional Engineering*.... 725
Entrained air—Effect on concrete. (Review) Bloem and Walker. *National Ready Mixed Concrete Association*.... 94
Equations for computing elastic constants from flexural and torsional resonant frequencies of vibration of prisms and cylinders. (Review) Pickett. *American Society for Testing Materials*.... 94
Evaluating the mechanical strength of coarse aggregates. (Review) Markwick and Shergold. *Journal of the Institution of Civil Engineers*.... 98
Experiments on reinforced concrete beams to determine the limiting reinforcement. (Review) Gthler, Ames and Friedrich. *Building Science Abstracts*.... 560
Experiments with admixtures for reducing the mixing water content and improving the workability of concrete. Kronsbein. (Review) *Zement*.... 96

F

FAILURES

Porous bodies under compression. (Review) Terzaghi. *American Society for Testing Materials*.... 97
Flexural and torsional vibration of prisms and cylinders computation of elastic constants. (Review) Pickett. *American Society for Testing Materials*.... 94
Fontana Dam—Aggregate production. (Review) McKamey. *Engineering News-Record*.... 270
Form linings for concrete surfaces. (Review) Pittman. *Engineering News-Record*.... 266
Forms—Steel—Chicago's filtration plant. (Review) *Engineering News-Record*.... 269
Freezing and thawing apparatus—Automatic accelerated. (Review) Wuerpel and Cook. *American Society for Testing Materials*.... 93
Frost beaming—Effect on pavement. (Review) Elgar. *Road Abstracts*.... 725
Further studies of the bleeding of portland cement paste. (Review) Steinour. *Bulletin 4, Research Lab., PCA*.... 557

G

Grouting—Multiple—Dam foundation (Review) Harza. *Engineering News-Record*... 267
—Pressure. (Review) Detig. *Beton u. Stahlbetonbau*.... 271
—Stabilising railway roadbed. (Review) Bonse. *Civil Engineering*.... 727

H

Hair cracks, crazing—In reinforced concrete pavement. (Review) Wooley. *Highway Research Abstracts*.... 562
Hardening of portland cement—Chemical processes. (Review) Hedin. Swedish cement and concrete Institute at the Royal Tech. University.... 561

I

Insurance against defective construction. (Review) Magnel. *Concrete and Constructional Engineering*.... 726
Investigational concrete pavements—Progress reports of cooperative research projects on joint spacing. (Review) *Highway Research Reports*.... 555

J

Joint spacing—Research. (Review) *Highway Research Reports*.... 555
Joints—Road and runway—Design of dowel bars. (Review) McFeeters and Hanrahan. *Concrete and Constructional Engineering*.... 725
—Weakened—Plane—Transverse—structural efficiency of. (Review) Sutherland and Casbell. *Public Roads*.... 102

L

Large triaxial testing machine built by Bureau of Reclamation. (Review) Blanks and McHenry. *Engineering News-Record*.... 265

LAWS AND PROPERTIES

Creep of concrete at overload. (Review) Shank. *Engineering Exp. Station News*.. 559
Hardening—Chemical processes in portland cement. (Review) Hedin. Swedish cement and Concrete Institute at the Royal Tech. University.... 561
Strength—Compressive—Effect of length of specimen. (Review) Tucker, *American Society for Testing Materials*.... 94
Strength, compressive—Effect of type of test specimen on apparent compressive strength of concrete. (Review) Mather. *American Society for Testing Materials Preprint*.... 93
Torsional resistance—Relation to tensile strength. (Review) Marshall. *Concrete and Constructional Engineering*.... 555
Lignosulfonate—Adsorption of—Portland cement dispersion. (Review) Ernsberger and France. *Industrial and Engineering Chemistry* 266
—Effect of adsorption on portland cement dispersion. (Review) Ernsberger and France. *Industrial and Engineering Chemistry*.... 93
Lime content in portland cement. (Review) Zollinger. *Zement*.... 556
Longeray Viaduct—Reconstruction. (Review) Turner. *Concrete and Constructional Engineering*.... 725
Lorraine Viaduct—Switzerland—Fixed end arch. (Review) *Concrete and Constructional Engineering*.... 553

M

MATERIALS AND TESTS

Admixtures—Effect on workability. (Review) Kronsbein. *Zement*. 96
—Vinsol resin—Effect of heat. (Review) *Tech. News. Bulletin, National Bureau of Standards*. 268
Aggregate—Coarse—Evaluating mechanical strength. (Review) Markwick and Shergold. *Journal of the Institution of Civil Engineers*. 98
—Production at Fontana Dam. (Review) McKamey. *Engineering News-Record*. 270
—Stone dust in. (Review) Arnstein. *Concrete and Constructional Engineering*. 93
—Influence of type and age on thermal-expansion. (Review) Keil and Tempel. *Zement*. 726
Calcium chloride—Wintertime treatment of pavements. (Review) Arnsfelt. *Statens Vaginstitut*. 731
Cement—Manufacture—Slurry rings in kilns. (Review) Gingrich. *Rock Products*. . . 95
—Natural—Blend improves concrete. (Review) Lovewell. *Engineering News-Record*. 265
Concrete—Freezing and thawing test—apparatus. (Review) *American Society for Testing Materials*. 93
—Mobile laboratory. (Review) *Concrete and Constructional Engineering*. 553
—Testing machine—Triaxial. (Review) Blanks and McHenry. *Engineering News-Record*. 265
Crushed materials—Process control—Sampling. (Review) Woods. *Rock Products*. . . 269
Mortar cubes—Temperature effect. (Review) Forbrich. *Rock Products*. 267
Portland cement containing Vinsol resin—Effect of heat. *Tech. News Bulletin, Bureau of Standards*. 268
—Dispersion by adsorption of calcium lignosulfonate. (Review) Ernsberger and France. *Industrial and Engineering Chemistry*. 266
—Hardening—Chemical process. (Review) Hedin. Swedish Cement and Concrete Institute at the Royal Tech. University. . 561
—Lime content. (Review) Zollinger. *Zement*. 556
—Manufacture—Clinker composition. (Review) Vera, Jr. *Rock Products*. 265
—Slurry sampling—automatic. (Review) Lintz. *Rock Products*. 265
—Paste—Bleeding. (Review) Steinour. *Bulletin 4, Research Lab., PCA*. 557
Measuring the consistency of concrete mixtures. (Review) Matern and Odemakk. *Statens Vaginstitut*. 727

MIXTURES

Consistency—Measuring. (Review) Matern and Odemakk. *Statens Vaginstitut*. 727
Proportions—Of concrete. (Review) Carneiro. *Road Abstracts*. 96
Mobile testing laboratory. (Review) *Concrete and Constructional Engineering*. 553
Multiple grouting of a dam foundation. (Review) Harza. *Engineering News-Record*. . . 267
Multi-story buildings of precast concrete. (Review) Lee. *Concrete and Constructional Engineering*. 554

N

Natural cement blend improves concrete. (Review) Lovewell. *Engineering New-Record*. 265
New method of revetting old man river. (Review) Pickett. *Engineering News-Record*. 266
Notes on controlling the uniformity of ready-mixed concrete. (Review) Walker. *Technical Information Letter No. 23*. 555

P

Patching of concrete pavement. (Review) *The Highway*. 558
Paved surfaces—Cracking—Soil movements. (Review) Elgar. *Road Abstracts*. 725

PAVEMENT

Bituminous—Measuring tar content. (Review) Arnsfelt. *Statens Vaginstitut*. 730
Highway—Patching—Shallow. (Review) *The Highway*. 558
Reinforced—Action of steel. (Review) Wooley. *Highway Research Abstracts*. 562
Rigid type—Joint spacing. (Review) Clemmer. *Highway Research Abstracts*. 273
Paving for the biggest planes in sight. (Review) *Engineering News-Record*. 266
Plastic flow—At high overload. (Review) Shank. *Engineering Exp. Station News*. 559
Poisson's ratio—Computations from flexural and torsional frequencies. (Review) Pickett. *American Society for Testing Materials*. 94
Portland cement—Dispersion—Adsorption of calcium lignosulfonate. (Review) Ernsberger and France. *Industrial and Engineering Chemistry*. 93
—By adsorption of calcium lignosulfonate. (Review) Ernsberger and France. *Industrial and Engineering Chemistry*. 266
Precast building units. (Review) Lee. *Concrete and Constructional Engineering*. 554
—Reinforced concrete cargo barges. (Review) *Concrete and Constructional Engineering*. 553

PRECAST UNITS

Barge ribs. (Review) *Concrete and Constructional Engineering*. 553
Building elements—Reinforced. (Review) Loesser. *Beton u. Stahlbetonbau*. 272
—Members. (Review) Lee. *Concrete and Constructional Engineering*. 554
—Parts—Prefabricated. (Review) Stoess. *Beton u. Stahlbetonbau*. 267
T-beams—Bridge construction. (Review) *Engineering News-Record*. 270

Prefabricated reinforced concrete structures. (Review) Stoess. *Beton u. Stahlbetonbau*..... 267
Prestressed concrete. (Review) Magnel. *Concrete and Constructional Engineering*..... 554
—(Review) Magnel. *Concrete and Constructional Engineering*..... 726
Pressure grouting. (Review) Detig. *Beton u. Stahlbetonbau*..... 271
Proportioning—Of concrete. (Review) Carneiro. *Road Abstracts*..... 96

R

Railway road beds stabilized with portland cement grout. (Review) Boase. *Civil Engineering*..... 727
Rapid determination of the bitumen content in pavements. (Review) Arnfelt. *Statens Vaginstitut*..... 730
Ready-mixed concrete—Control of uniformity. (Review) Walker. *Technical Information Letter No. 23*..... 555
Reinforced concrete chimney base at Bradford. (Review) Manning. *Concrete and Constructional Engineering*..... 553

RESEARCH

Pavement—Joint spacing. (Review) *Highway Research Reports*..... 555

RESERVOIRS AND TANKS

Water storage—Two-way prestressed. (Review) Carr. *Engineering News-Record*..... 271
Responsibility for defective construction. (Review) Magnel. *Concrete and Constructional Engineering*..... 726
Revettment—Reinforced concrete mats. (Review) Pickett. *Engineering News-Record*..... 266
Rigid type pavement and joint spacing. (Review) Clemmer. *Highway Research Abstracts*. 273
Rings in cement kilns, part 1: Why they form and how to prevent them. (Review) Gingerich. *Rock Products*..... 95

S

Salt treatment—Wintertime—Damage on concrete pavement. (Review) Arnfelt. *Statens Vaginstitut*..... 731
Scientific sampling for accuracy. (Review) Woods. *Rock Products*..... 269
Shallow patching adds years to life of concrete pavements. (Review) *The Highway*..... 558
Shasta Dam spillway—Vacuum processing. (Review) Rippon. *Engineering News-Record*. 270
Shear reinforcement in beams. (Review) Ashdown. *Concrete and Constructional Engineering*..... 725
Slurry rings—Formation and prevention. (Review) Gingerich. *Rock Products*..... 95
—Sampling methods. (Review) Lintz. *Rock Products*..... 265
Soil cement in aerodrome construction. (Review) Andrews. *Journal, Engineers Australia*..... 729
—Movements as affecting paved surfaces. (Review) Elgar. *Road Abstracts*..... 725
Special steel forms speed concreting at Chicago's water filtration plant. (Review) *Engineering News-Record*..... 269

SPECIFICATIONS

Building codes—Brazilian and U. S. compared. (Review) Boase. *Engineering News-Record*..... 273
Stone dust in aggregate. (Review) Arnstein. *Concrete and Constructional Engineering*..... 93
Stress conditions for the failure of saturated concrete and rock. (Review) Terzaghi. *American Society for Testing Materials*..... 97
Stresses in beams near supports. (Review) Forbrich. *Rock Products*..... 267
Structural behavior of Fontana Dam revealed by dual-purpose instruments. (Review) Waugh. *Engineering News-Record*..... 268
Structural efficiency of transverse weakened-plane joints. (Review) Sutherland and Casbell. *Public Roads*..... 102
Studies of concrete with entrained air. (Review) Bloem and Walker. *National Ready-Mixed Concrete Association*..... 94

T

Tanks—Water storage. Two-way prestressed. (Review) Carr. *Engineering News-Record*.... 271
Temperature effect on mortars. (Review) Forbrich. *Rock Products*..... 267

TESTS AND FIELD CONTROL

Mixes—Entrained air and mixing water requirements. (Review) Bloem and Walker. *National Ready Mixed Concrete Association*..... 94
Proportioning of concrete. (Review) Carneiro. *Road Abstracts*..... 96
Ready-mixed concrete—Controlling uniformity. (Review) Walker. *Technical Information Letter No. 23*..... 555
Specimens—Effect of length on strength. (Review) Tucker, *American Society for Testing Materials*..... 94
Specimens—Effect of type on compressive stress. (Review) Mather. *American Society for Testing Material*..... 93

TESTS OF MEMBERS & COMPLETED STRUCTURES

Beams—Compression test—Field evaluation. (Review) Diggory. *Concrete and Constructional Engineering*..... 554
—Limiting reinforcement determination. (Review) Gthler, Amos, and Friedrich. *Building Science Abstracts*..... 560
—Prestressed—Measurement of creep and buckling. (Review) Magnel. *Concrete and Constructional Engineering*..... 55
—Twisted bars. (Review) Hajnal-Konyi. *Structural Engineer*..... 100

Dams — Fontana — Dual-purpose instruments. (Review) Waugh. *Engineering News-Record*..... 268
Pavement sections—Joint spacing. (Review) *Highway Research Reports*..... 555
Tests on square twisted steel bars and their application as reinforcement of concrete. (Review) Hajnal-Konyi. *Structural Engineer*. 100
Theory of elastically restrained beams, applied to statically indeterminate reinforced concrete beams. Hilal. *Costa Tsoumas & Co. Press*..... 95
The influence of aggregates type and of age on the thermal-expansion of concrete. (Review) Keil and Tempel. *Zement*..... 726
The Lorraine Viaduct, Switzerland. (Review) *Concrete and Constructional Engineering*..... 553
The Proportioning of Concrete. (Review) Carneiro. *Road Abstracts*..... 96
The reconstruction of the Longeray Viaduct. (Review) Turner. *Concrete and Constructional Engineering*..... 725
Thermal expansion—Influence of aggregates type and age. (Review) Keil and Tempel. *Zement*..... 726
The torsional resistance of plastic materials with special reference to concrete. (Review) Marshall. *Concrete and Constructional Engineering*..... 555
Torsional resistance of concrete. (Review) Marshall. *Concrete and Constructional Engineering*..... 555
Triaxial testing machine—Bureau of Reclamation. (Review) Blanks and McHenry. *Engineering News-Record*..... 26
Turbidity tests—Dispersion by adsorption of calcium lignosulfonate. (Review) Ernsberger and France. *Industrial and Engineering Chemistry*..... 93
Twisted reinforcement—Application of. (Review) Hajnal-Konyi. *Structural Engineer*..... 100
Two-way prestressed concrete water storage tank. (Review) Carr. *Engineering News-Record*..... 271

V

Vacuum processing of Shasta Dam spillway. (Review) Rippon. *Engineering News-Record*. 270
Vebe-apparatus—Measuring consistency. (Review) Matern and Odemakk. *Statens Vaginstitut*..... 727
Vinsol resin—Portland cements—Effect of heat on. (Review) *Technical News Bulletin Bureau of Standards*..... 268

W

Wintertime concreting at the Soo. (Review) *Engineering News-Record*..... 268
Workability—Effect of admixtures. (Review) Kronsbeim. *Zement*..... 96

Y

Young's modulus—Computation from flexural and torsional resonant frequencies. (Review) Pickett. *American Society for Testing Materials*..... 94